DESIGN OF BUILDING FOUNDATIONS

A THESIS

Submitted as partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE IN CIVIL ENGINEERING.

By

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Six Raymond Pile-drivers installing 8000 Raymond Concrete Piles for The United States Internal Revenue Building, Washington, D. C.

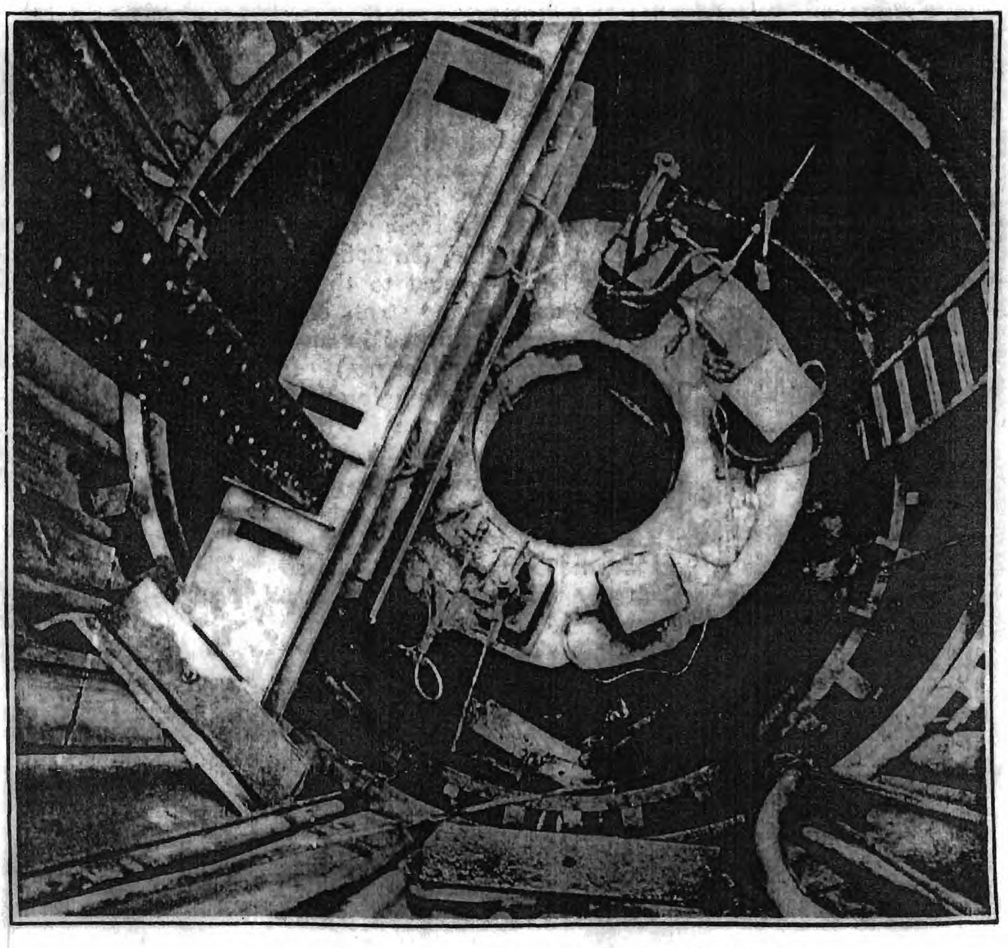


FIG. 2—FOUNDATION SHAFT WITH JACKING RIG

Part of frame removed, showing jacks on concrete ring at head of 6-ft. shaft. Top of shaft 11 ft. 3 in. diameter inside steel sheeting. One of the four jacking posts is shown at the left.

Introduction.

One of the most newly developed fields in the science of engineering is that of Soil Mechanics, which treats of the science of soils as applied to sustaining engineering structures. I have studied very carefully to the best of my ability the various literature on this subject and have in the best manner I could, in so short a space, applied it to the design of building foundations. My references have been to the bibliography included in the back of this thesis, and for further references I refer you to "The Bibliography of Physical Properties and Bearing Value of Soils," compiled by Morris Schero, Proc. Am. Soc. C. E. v57 No.6, pp871-921.

I am greatly indebted to the following persons for there assistance in the writing of this thesis: Captain I. L. Daniels, Constructing Engineer, U. S. Treasury Department, Professor F. C. Snow, Head of the Department of Civil Engineering, Mr. H. F. Wilds, The Coca-Cola Company, Professor J. H. Lucas, Department of Civil Engineering, Mr. R. G. Lose, Consulting Engineer, and to my Father, Mr. J. E. Futral, Central of Georgia Rwy.

I have attempted to make no integral part of this thesis totally complete, except the design of footings for building foundations. In the related sections, I have merely endeavored to stress the most important facts relating to the particular subject.

Foundations in General

A foundation is that portion of the substructure whose function is the distribution of the load to the earth. The perfect foundation for an engineering structure is bed rock with a high compressive strength and which will not disintegrate or weather when exposed to the elements. In building foundations the first thought to try a reach bed rock and rest the structure upon it. However this is rarely possible because bed rock is often so far below the surface of the ground that to reach it would be very difficult and is economically prohibited. Substantial bed rock will sucessfully carry any load that can be imposed upon it by a concrete structure.

If we find it impossible to reach bed rock, then the next step is to try the adaptability of spread footings to the case. The function of a spread footing is to distribute the concentrated load of the column or wall over an area such that the resisting or safe bearing power of the soil will not be exceeded. Load tests are usually made on the soil to determine the safe bearing power of an area approximately the size of the proposed footing. From this information the exact size of the spread footing is then determined.

In some cases the soil has a very low bearing power and it would be uneconomical to place the footings at such a depth and make them of such dimensions as would be required to support the structure. In this case piles or piers are used to transfer the load to a lower strata that can satisfactorily resist it. Piers are usually used where the structure is of considerable magnitude and rock is not very far below the surface of the ground. Piers may be sunk by open pits or caissons depending upon the conditions encountered. Lately a great development and improvement has taken place in the methods and equipment used in sinking open caissons; so that pneumatic caissons are being almost entirely replaced except in the case of monumental structures.

Examination of the Location.

The examination of the foundation soil, on which a minor engineerign structure is going to rest, should extend to a depth at least equal to the width of the largest footing. In a structure of major importance and magnitude extensive investigation of the site should be made. Too much emphasis cannot be placed upon the importance and necessity of examining the site of the proposed foundation thoroughly and completely even before the site is bought. A small expenditure in this manner may save much later on.

Test Pits.

This is the simplest, oldest and most costly method of examining the soil strata and consists of simply digging open pits to the depth desired. This method is good because it allows the foundation material to be examined in situ. At least one test pit is usually sunk for each foundation area upon which an engineering structure of any importance is to be placed. The remaining foundation area is then investigated by probings or borings. Test pits allow the strata to be examined in its natural position; therefore its compactness and water bearing qualities are directly observable. It also allows test loads to be applied at the bottom of the pit. Test pits are very expensive if carried to a depth exceeding 10 to 15 feet, depending upon the soil, and are practically impossible after water is encountered.

Soundings or the Rod Test.

Examination by soundings or the rod test may be made on soft soils to a depth of from 20 to 30 feet, by driving a solid bar of tool steel 5/8 to 7/8 inch in diameter, depending upon the material to be penetrated. The rod is driven in sections. The bottom section is pointed at one end and threaded at the other. If the earth is soft so that the bar can be churned down 5 or 6 feet by hand, the bottom section of the rod should be made about 10 feet in length. Additional sections about 5 to 7 feet in length and threaded at both ends should be provided in sufficient quantity to reach the depth desired. After the rod has been worked down as far as possible by hand, it is then driven with a 10 or 12 pound sledge hammer. The top of the rod should be provided with a threaded protective cap while it is being driven, inorder to protect the threads of the rod from injury. Using 3 men on this work it is possible to sound from 200 to 300 linear feet per eight hour day. An accurate record is kept of the number of blows required to drive the rod down each successive foot. The nature of the strata is determined by the action of the rod in driving. If a rock is encountered, there will be a sharp, springing rebound. When it is thought that rock has been encountered, numerous soundings should be made in the immediate vicinity to determine whether a boulder or ledge rock has been reached. One should be particularly careful to distinguish be tween a boulder and bed rock if the structure is to be a heavy one or a bridge pier located in a stream which is liable to scour. In pulling the rod, a chain and lever, and block and tackle are used.

Borings.

The two types of borings are dry borings and wash borings. Dry borings are usually made with an earth auger to a depth of from 50 to 100 feet, and gives the most accurate results of any type of boring. However, its field is limited to the type of soil which will remain in the helix of the auger until brought to the surface. This quality is usually found in soils which contain clay and are cohesive. A post hole auger may sometimes be used for examining the soil to a small depth, and is especially valuable in examining the soil to a depth of 5 or 6 feet below the bottom of proposed footings, after the excavations have been made. The nature but not the compactness of the soil may be determined by borings made in this manner, as the soil is compressed somewhat when cut off or partially sheared from the surrounding soil by the helix of the auger. A derrick or hoist is usually used in making these borings as in well digging. If sand, gravel, or any such non-cohesive soils or encountered, then the hole must be cased. This is done by driving a pipe or casing of large enough diameter that the auger bit and rods will freely pass through it.

The wash boring method for making soil test is almost always used for deep and difficult foundations and if properly interpreted will successfully show the nature of the soil to a depth of from 120 to 200 feet. The outer pipe or casing is driven by inserting a smaller pipe inside of it, and forcing water down the inside pipe. The soil and water will flow up between the two

CORE DRILL

pipes. Samples of practically undisturbed soil may be obtained at intervals by withdrawing the jetting pipe and driving a pipe into the undisturbed soil beneath the bore hole, and withdrawing the sample with the pipe.

Drilling.

When the subsoil is composed of very hard soil or rock, it may become necessary to use a percussion drill along with some form of core drill. A shot drill is usually used in taking cores. However, when the rock is very hard and greater speed is desired, a diamond drill may be used. The cores, as taken, should be filed away in proper sequence and properly identified. In using a percussion drill great care must be taken, that a strata hard enough to support the structure is not passed by unnoticed. To prevent this, cores should be taken at frequent intervals. In using a core drill, as in all soil investigations, great care must be used to distinquish between boulders and bed rock.

The Science of Footings.

General Statement.

In this discussion I shall endeavor not to try and make the science of soils and foundations an exact one, because at the outset I realize the utter futility of this proceedure. The science of the bearing power of soils can never be exact, because of the very nature of the materials involved. If the soil has been prepared by man, so that he knew its exact constituents, how it had been placed, and and the pressure placed upon it, in its consolidation, then it would be comparatively easy to determine and calculate its safe bearing power. However, this is far from the actual case, since these materials have been placed by the forces many eons ago of which we have meagre record as to the process. Sometimes they were placed under enormus pressures, sometimes by gentle sedimentation, and always under conditions which produced infinitely varied strata. Many laboratory tests have been made on foundation soils, but I wish to call your attention to the fact that when the soils are disturbed or moved from their native bed, then, some of the soils most important qualities used for determining their suitability for foundation purposes have been altered. For this reason, I shall have little to say of laboratory tests, although they are very helpful when accompanied by load tests and are useful in determining the soil type and its physical characteristics. I seriously believe that their can never be any great general application of formulaë to soils as an exact science. Every soil is a different case, for in every case there is a different arrangement of more or less natural cementing material and soil particles, and there is no way of telling the pressure exerted upon these materials in consolidating them. We cannot disturb this natural arragement and carry the soil in the laboratory and make accurate load tests upon it to determine its bearing power. This must be done in the field. To disturb soil and test it in this manner is like pulverizing a concrete cylinder, then running a compression test on it and saying we have its true compressive strength. The laboratory tests which are very helpful in determining are the tests for (1) Specific Gravity and Density of Structure, (2) Smoothness of the Grain Surfaces, (3) Intensity of Capillary Pressure or Cohesion.

Bearing Capacity of Soils.

The bearing capacity of soils is dependent upon (1) the composition of the soil, (2) the amount it is confined, (2) the amount of moisture contained.

Tests or borings should be made in order to ascertain the types of soils upon which the foundation is to rest. Test pits should be opened to the depth of the bottom of the foundation and load tests made.

Factors not to be over looked: (1) A small area will bear a larger load per unit of area for a short time than a larger area will indefinitely. (2) Ordinary soils will bear more unit load the greater the depth, due to the fact that they become more condensed from the weight of the superimposed load.

(3) The factor of depth with clay is especially important, because the greater the depth below the ground, the less is the liability of its being displaced laterally due to other excavations in the immediate vicinity; also the greater the depth, the less the variation in the moisture content.

(4) The bed of the foundation should always be placed below the frost line.

Practically any rock in its native bed will sustain any load that can be placed upon it by concrete. In preparing a rock bed, all loose and decayed portions should be cut awayaand the bed dressed to a plane surface as near perpendicular as possible to the resultant of the pressure. Any fissures or seams in the rock should be grouted. A sloping bearing surface should be stepped.

Elasticity of Soils.

Soils have certain definite physical properties which affect their bearing power and therefore, their degree of usefulness in carrying foundation loads. First soils have definite, although imperfect physical elastic properties. The tests made on the Atlanta, Georgia Post Office at a cost of $500.00 and shown on Plate No. 2 give curves illustrating the compression, then the rebound or recovery of the soil. Another fine example of the elastic properties of soil is the case of a well seasoned railroad embankment, where the cross-ties settle 3/8 of an inch under locomotive and car loads and yet recovers under millions of applications of these loads.

When a relatively small area is loaded in a sandy soil, a certain amount of the soil flows from under the loaded area's edge, relieving the pressure along the perimeter of the footing. Vibration in this building will greatly increase the flow and should be given proper consideration. See Bibliography in back of this thesis. While this flow is occuring along the edges, the center grains are bein g locked in place, aqquiring a large intensity of pressure and acting as elastic columns of loosely built up rubble masonry. After some settlement has occured, the soil becomes consolidated, the flow of soil ceases, and the supporting ground be haves like an elastic solid, until the pressure becomes so great that it exceeds its yield point and partly breaks down the formation of the pressure bulb. Now, a second flow occurs with increased settlement.

Relation of Area to Bearing Capacity.
The ultimate bearing capacity of a soil is the load greater than which, the settlement of a given bearing area becomes progressive.

If the soil pressure under a footing has a more or less parabolic distribution as we know it has, then, we know that the bearing capacity of such a soil cannot be directly proportionate to the loaded area. We do know that the bearing power per unit area becomes less the larger the loaded area. The relation between the area loaded and the settlement depends principally on the co-hesion or actual shearing strength of the soil. For soils having great cohesion, the settlement produced by a given unit load increases almost in direct proportion with the diameter of the loaded area. For perfectly cohesionless soils, the size of the area matters but little.

Effect of Depth on Bearing Power.
With increasing depth from the surface of the ground, the settlement produced by a given unit load decreases. This is obvious because the deeper you go the more superimposed weight is that strata carrying and has thereby been more consolidated than the stratum nearer the surface. Also because of the load of the upper stratum and the confinement, there is less lateral and vertical movement of the soil. The greater the cohesion of the soil, the smaller is the effect of thedepth in decreasing the settlement for a given unit load. This is naturally true because nature has already caused a greater consolidation than in cohesionless soils.

Effect of the Degree of Permeability and Time on Settlements.
Every soil without exception is compressible under pressure, the pressure tending to produce a decrease in volume. Every soil consists of individual grains separated by spaces called voids. If these voids are filled with air, then the volume change can take place readily, because the excess air has but to escape toward the surface. However if the voids are completely filled with water as is usually the case with every fine grained and very compressible soils such as clay, silts, and mud; a decrease in volume necessitates a considerable decrease in the water content. We must remember that water is relatively incompressible and that the settlement cannot progress with a greater speed than the squeezing out of the excess water.

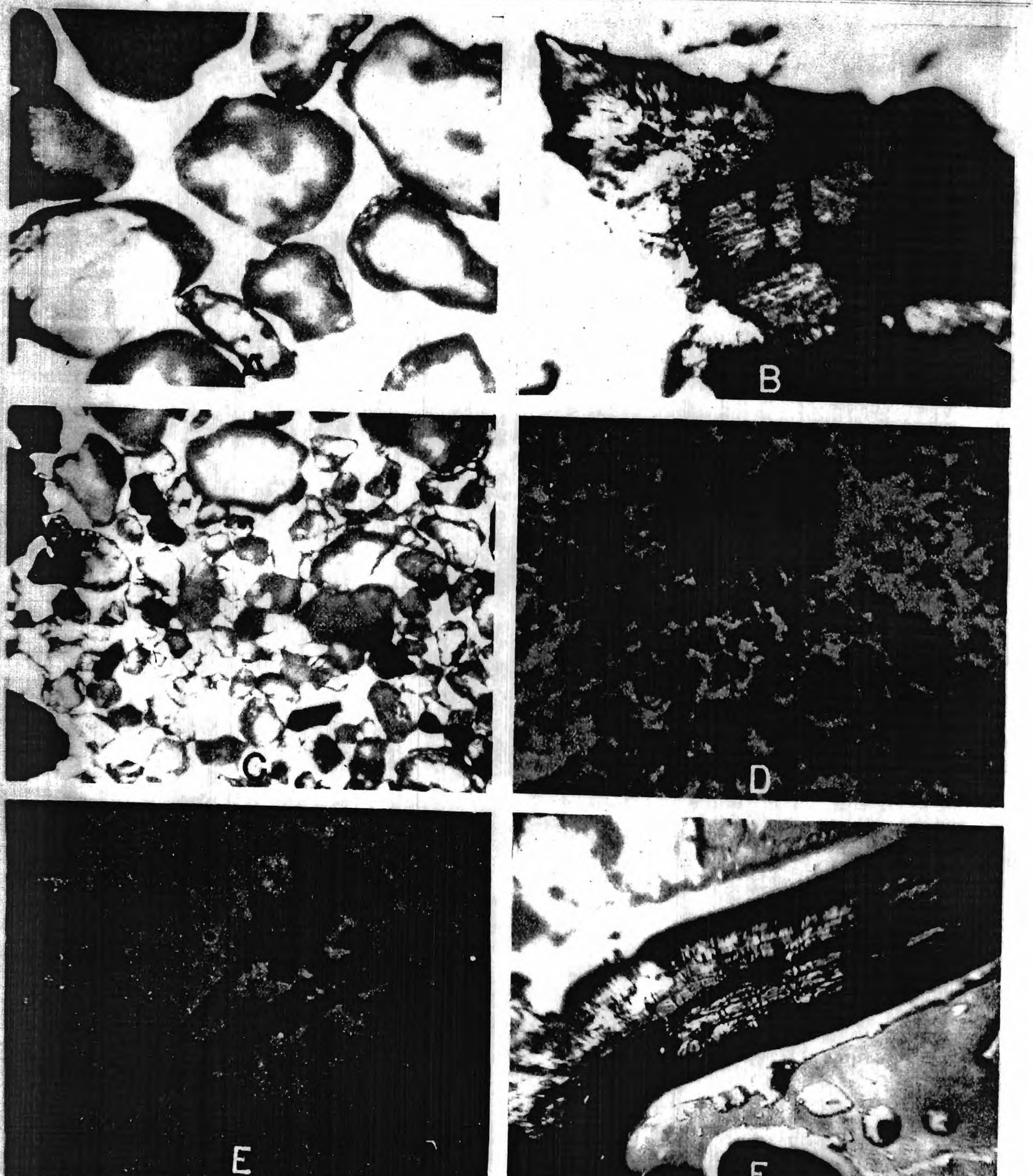


FIGURE 30.—PHOTOMICROGRAPHS OF SOIL CONSTITUENTS. A.—BEACH SAND. B.—ANGULAR SAND GRAIN. C.—GLACIAL SAND. D.—SOIL CONTAINING DIATOMS. NOTE SPONGELIKE APPEARANCE. E.—SINGLE DIATOMS. F.—PEAT-BOG MATERIAL. NOTE FIBROUS STRUCTURE AND FILM OF WATER SURROUNDING INDIVIDUAL PARTICLES

The less permeable the soil, the greater resistance offered to the escape of this water and consequently the slower the settlement. For this reason, the settlement of many foundations will not occur at once, but will lag an amount of time dependent upon the permeability of the soil. The usual time required for the excess water to be squeezed out of silts and coarse grained mud should be 2 or 3 years, while for some clays it would take perhaps one hundred years before the water would be completely squeezed out.

Settlements may be divided into two general classes, (1) Those due to lateral flow of the soil with little or no consolidation and (2) those due to lateral flow and consolidation combined.

The properties of sand and clay are important factors to deal with in the construction of foundations and for that reason, I give below these properties as defined by Dr. Terzaghi.

Clay.

1. Volume of voids may be as high as 98 per cent of total.
2. Shrinks in drying.
3. Has marked cohesion, depending upon moisture content.
4. Is plastic.
5. Compression very slow when load is applied to the surface.
6. Is very compressible.

Sand

1. Volume of voids is about 50 percent of maximum.
2. Does not shrink in drying.

3. Has negligible cohesion when clean.

4. Is not plastic.

5. Compresses almost immediately as the load is applied to the surface.

6. Is far less compressible than clay.

Causes of Failure in the Bearing Power of Soils.
By the Committee on Soils, Am. Soc. C. E.

Bearing Power becomes deficient because of.

1. Compression
- (a) From unequal loading within the elastic limit.
- (b) From loss of cohesion.
- (c) From crushing edges of the grains.
- (d) From shrinkage of organic matter.
- (e) From loss of water content.

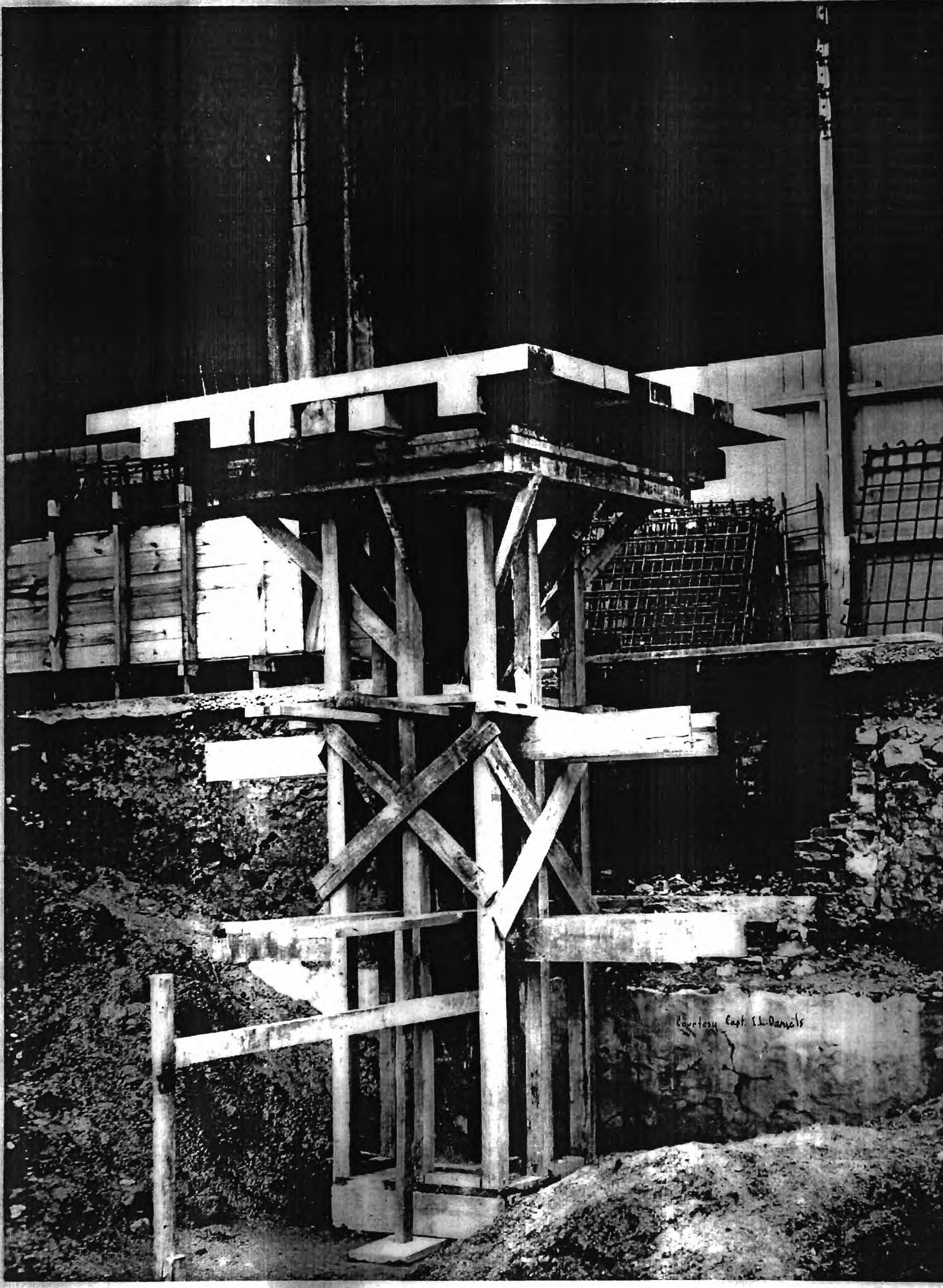
2. Flowing
- (f) From saturation.
- (g) From lack of cohesion under weight and pressure.
- (h) From exceeding cohesive strength.
- (i) Vibration of structure.

3. Sliding
- (j) From sliding of bodies of material on an underlying and usually inclined layer of lubricating material.
- (k) From sliding of material previously immersed in water when water level is quickly lowered.
- (l) From sliding of the structure on the soil.
- (m) From sliding of the structure together with soil.

4. Erosion
- (n) From flowing water and the fluctuation of th e water table.
- (o) From wind.
- (p) From weathering and frost.

5. Chemical changes.
- (q) Possible chemical influences.

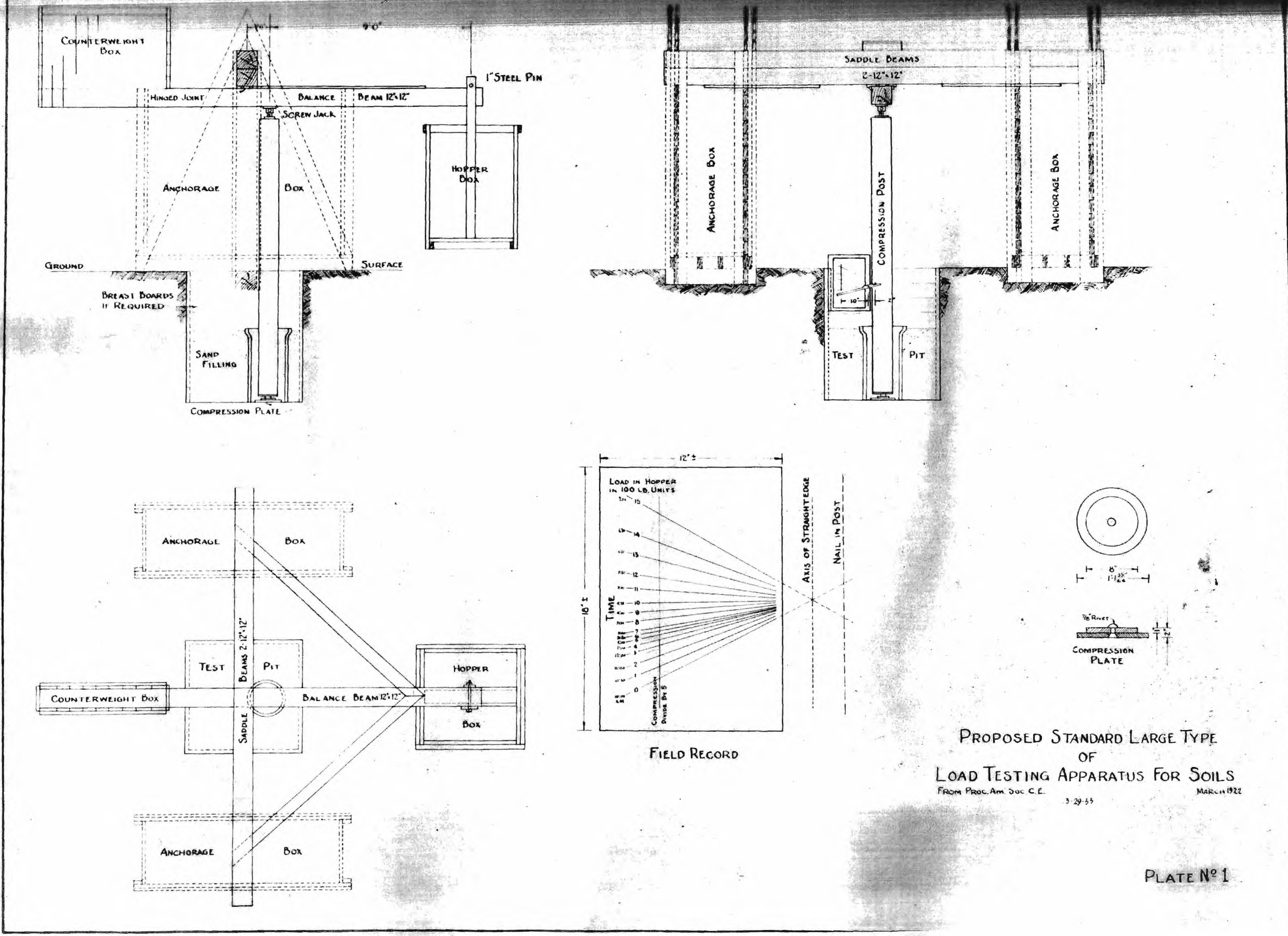
Load Tests on Soils.
I have included in this thesis, through the courtesy of Capt. I. L. Daniels, the data on the load tests made at the site of the New Atlanta Post Office. The diagrams plotted from the data obtained in these tests demonstrate three characteristics of soils, previously discussed: (1) The elastic property of rebound, (2) That the large r the area in cohesive soils, the greater the settlement per unit load. (3) The progressive settlement after loading has ceased shows water drainage and permeability.



Load Testing Apparatus

LOAD TESTING APPARATUS LOADED
81000 LBS. 3'x3' AREA

Courtesy Mr. H. F. Wilds



Proposed Standard Large Type of Load Testing Apparatus for Soils

From Proc. Am. Soc. C.E. March 1922

3-29-33

Plate No 1

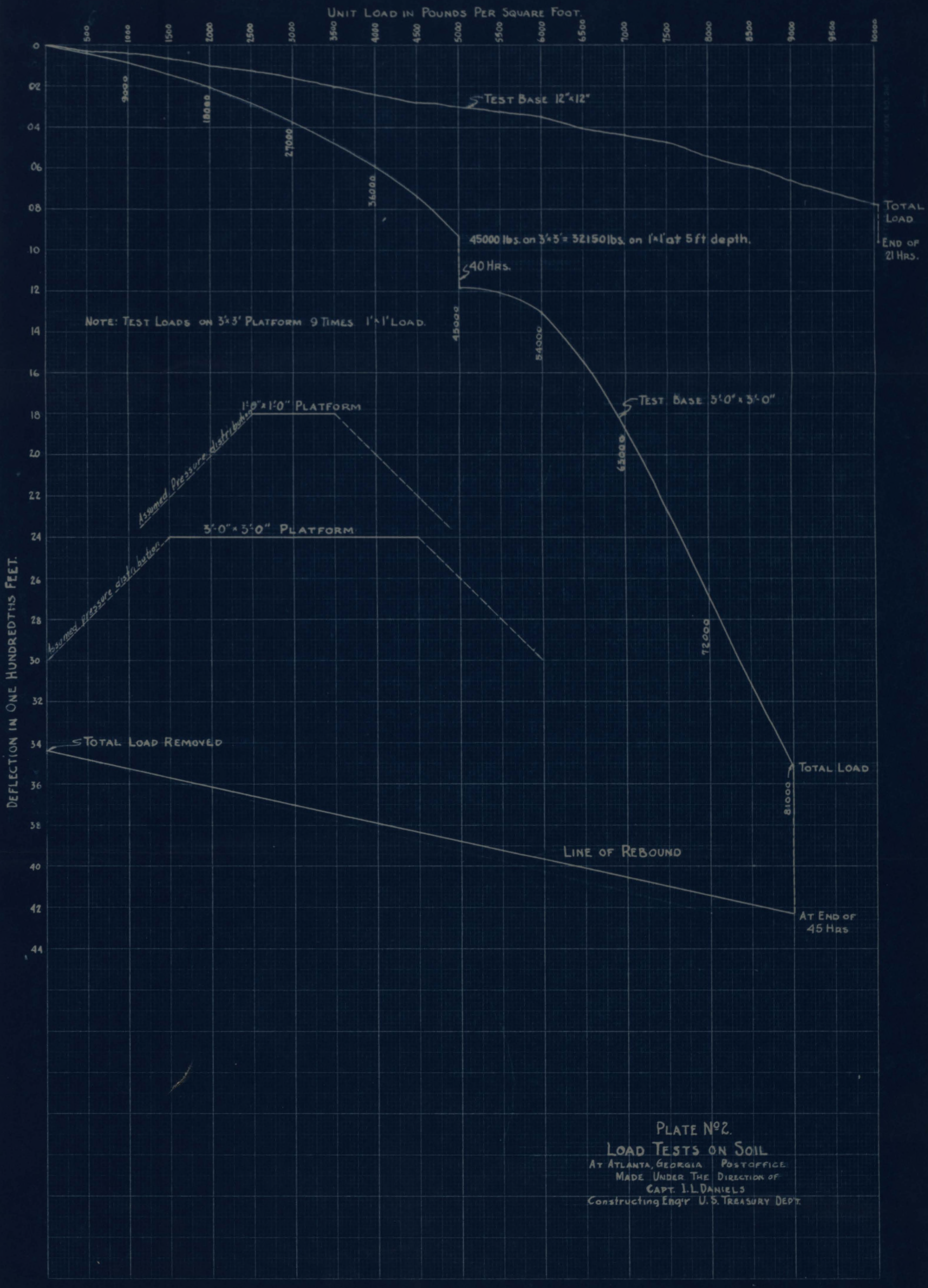


Plate No 2.
Load Tests on Soil
At Atlanta, Georgia Postoffice
Made Under The Direction of
Capt. I. L. Daniels
Constructing Eng'r U. S. Treasury Dept.

Making Load Tests.
The Committee on Soils of the Am. Soc. of C. E. have designed a practical field testing apparatus as shown by Plate No. 1. This apparatus is claimed to have a capacity of about 10 tons per square foot and a degree of sensitiveness such aswill give a compression diagram for soils o f very small compressive strength. It is claimed to be very economical to build and operate, requiring only cheap ordinary materials for construction and utilizing such readily available materials as earth and water for weights. The laboratory tests which show the characterists of soils which are helpful in engineering work are the specific gravity and density of structure, smoothness of the grain surfaces, and intensity of capillary pressure or cohesion. In making the load tests careful record should be made of the settlement at each increment of load and it should be notice when the settlement of the soil becomes progressive. This is done by loading the soil up to about 3 to 4 tons per square foot letting it rest 24 hours and noticing the settlement and loading againetc. until the point of progressive settlement is found. From the data obtained graphs are drawn.

Footings.

Footings in General.

The requirements in any footing is first to make the settlement as small as possible and second, if settle- should occur, it should be uniform for the whole structure. This may be accomplished by first determining the safe bearing power of the soil for a footing near the size of the ones build. Uniform settlement may be secured by designing the footings in such a way that the pressure on the soil is very nearly uniform throughout the entire structure. The common types of reinforced concrete footings are: 1.Wall Footings, 2.Independent Column Footings, 3.Combined Footings, 4.Cantilever or Strap Footings, 5. Raft Foundations. Footings are designed to carry the upward reaction of the earth or the piles supporting it.

Wall Footings.

A wall footing as a rule consists of a slab projecting the required distance on each side of the wall to furnish an area such that will support the load placed upon it. These projections act as cantilever beams. For small footings it is more economical to use plain concrete. If however, the projections are large it is cheaper to use reinforced concrete. In calculating the bending moments, each portion should be considered as a cantilever beam with the critical section at the face of the wall. The maximum bending moment is given by the equation, $M = \frac{1}{8}w(L - a)^2$ where L is the width of the footing, a is the width of the wall and w is the unit soil pressure. The bond stresses are calculated for the section at the face of the wall as the critical section and just as for ordinary beams. The section at d distance face of the wall (d being the distance from the top of the footing to the center of the reinforcing bars) is taken asthe critical section for vertical shearing stress. The reinforcing determined in the usual manner consists of bars placed at the bottom of the footing,allowing 3 in. for protection, at right angles to the wall. Special care should be given the calculation of the bond stresses. Due to difficulty in placing and impaired effectiveness it is preferable to keep the stresses below the values which require web reinforcing.

Design of a Wall Footing.

A 20 in. wall supports a total load of 32,500 pounds per linear foot, and rests on a soil whose safe bearing power is 3,500 pounds per square foot. Consider that we are using a concrete F'_c = 2000 lb. per sq. in. Design a footing to support this wall using the Joint Committee Specifications. Assume weight of footing = 2300 lbs./ ft.

Total width of footing required is 34,800/3,500 = 10 ft.

The maximum moment = $M = \frac{1}{8}w(L - a)^2$ $\quad$ $w = 34800/10 = 3480$ lbs.

$M = 3480(10 - 1.66)^2 \times 12/8 = 363,000$ in. lbs.

Use allowable unit stresses of $f_s = 18000$ lbs. per sq. in.

$f_c = 800$ lbs. per sq. in. and $K = 139$, $j = .867$, and

$n = 15$ (see Table No. 2)

$d = \sqrt{M/Kb} = \sqrt{363,000/ 139 \times 12} = 14.7$ use 15 in.

The J.C.S. specifies 3 in for protective coating. See J.C.S. section 67. See Plate No. 3, Fig. No. 1.

$d + d' = 15 + 3 = 18$ in.

Check the assumed weight: $18 \times 12 / 144 \quad \times 150 = 2250$ lbs.

$A_sc = M/f_sjd = 360000/18000 \times .867 \times 15 = 1.6$ Sq. in. per foot. See section 197 J.C.S. Use deformed bars intermediate grade steel. $u = .035 f'_c = .035 \times 2000 = 75$.

Summation $\phi = V/ujd$, where V equals the total shear at the section in pounds.

Summation $0 = 4.165 \times 3480/75 \times .867 \times 15 = 14.75$ in. per ft.

The above requirements are satisfied by using 1 in. sq. bars spaced 3 in. center to center. See Table No. 1.

Now investigating for diagonal tension, the shear "d" distance from the face of the wall is: $32500/10 \times 25/12 = 6770$ lbs.

And the unit shear which is a measure of diagonal tension is: $v = V/bjd = 6770/12 \times .867 \times 15 = 43$ lb. per sq. inch. This is well within the allowable limit which is $.03 \times 2000 = 60$ lb. per sq. in.

Column Footings.

In General.

Footings are designed for the upward pressure of the soil or the upward reaction of the piles supporting it. We know definitely from tests performed that the column load is not distributed evenly over the entire footing. However, if the footing is placed in an elastic or compressible soil the probability of obtaining a uniform load is greater than when placed on rock. In the elastic soil, at first a large portion of the load is supported by the soil directly under the column. This produces a small settlement of the footing directly under the column. If the column were not connected with the remainder of the footing, this settlement would continue until the soil was compressed sufficiently to raise its bearing power to the point where it would support the applied load. Since the column is connected with the footing, the incipient downward movement of the section directly under the column produces shear in the adjacent sections and forces them down, although to a slightly less degree, because of the bending produced in the footing slab. The amount of settlement is less at the edge that at the center of the footing, therefore, the footing assumes the shape of a curve of the same nature as would have been produced if the reactions of the ground were active pressures carried by the column as a support. It is not necessary for the footing to be moved upward through the column to say that it has failed. It may fail because the footing can not withstand excessive settlement.

Various formulas and means have been derived for calculating the bending moment in column footings, but many of them have wide variations, and it has been found that Professor A. N. Talbot's formula which he developed at the University of Illinois and as given in his Bulletin No. 67, is the simplest and most accurate method. After making tests on many footings reinforced in various ways, he studied very carefully the flexure curves and came to the conclusion that all the upward load on the rectangle lying between the face of the pier and the edge of the footing is considered to act at a point half-way out from the pier, and half of the load on the two corner squares is considered to act at a center of pressure located at a point 0.6 of the width of the projection from the given section. This gives the formula $M = \frac{1}{2}w(a + 1.2c)c^2$. This formula will not apply to any but square footings. The bending moment of rectangular footings is calculated by dividing them into rectangles or trapezoids tributary to the sides of the column, using the distance to the actual center of gravity of the area as the moment arm of the upward force. Bond stress is one of the most important factors in the design of footings and should be closely watched as it often governs in the calculating of the reinforcing. The critical section for diagonal tension in a column footing is at the face of the square formed by measuring "d" distance out from the face of the column. The unit shear is checked to see that it is within the allowable limit as specified by the Joint Committee. Punching shear which is the tendency of the column to punch its way through the footing, and it must be overcome by the shearing resistance of the concrete. The shearing resistance is equal to the shearing strength of the concrete on an area equal to the effective depth of the footing multiplied by the perimeter of the column. The value of the unit shearing stress used in calculating the resistance to punching shear is equal to $.06f'_c$. Punching shear usually governs the depth of footings. Footings Should Be: 1.Strong enough to resist the reaction of the soil. 2.Rigid enough to prevent excessive deflection under load. 3. Concentric with the applied load. 4. The bottom should be below the frost line. In order to make the footing concentric with the applied load, the footing is built so that its center of gravity will coincide with the theoretical point of application of the load.

Design of Independent Footings.

Procedure: 1. Determine the area of the base of the footing by dividing the column load, plus the assumed weight of the footing, by the safe allowable unit bearing pressure on the soil.

2. Determine the area of the pedestal,if any.

3. Determine the depth of the footing required by punching shear, using the column load exclusive of the weight of the footing.

4. Select the type of footing to be used, i.e., whether single slab, stepped, or sloped.

5. Compute the diagonal tension to determine whether or not the assumed thicknesses at the various points are sufficient.

6. Compute the bending moments and the required amounts of steel. In stepped footings, this is done at every step.

7. Check the bond stresses in the steel to make certain that they are with in the allowable limit.

<u>Design of a Two-way Reinforced Block Footing.</u>
A column 24 in. square supports a load of 300,000 lbs. Design a single slab, reinforced in two directions, to support this load on a soil whose safe bearing power was found to be 4000 lbs. per square foot. Assume a concrete having f'_c = 2000 lbs. per square inch. See Plate No. 3 Figs. 2
Assume the weight of the footing as 31000 lbs.
The bearing area required = 331000/4000 = 82.75 sq. ft.
We use a base 9 ft. 3 in. square furnishing an area of 85.56 square feet.
The unit soil pressure, w, due to the column load = 331000/85.56 = 3850 lbs. per square foot.
Calculating the Depth for Punching Shear:
Area X = L x L = L^2, Area Y = a x a = a^2
w = unit soil pressure, z = perimeter of column, d = depth of footing. u_p = unit punching shear = .06 f'_c.

$$d = (X - Y)w/u_p \times z \qquad d = \frac{(81 - 4) \times 3850}{4 \times 24 \times 120} = 25.8 \text{ use 26 in.}$$

Using a 3 in. protective coating, d + d' = 26 + 3 = 29 in.
Checking the assumed weight of the footing, we find the weight to be: 85.56 x 29 x 150/12 = 31,000 lbs.
Check for Diagonal Tension:
The total net upward pressure outside of a square concen-

tric with the column, and each side being "d" distance or 26 in. in this case from the corresponding face of the column = V.

$$V = w\left(L^2 - \frac{(2d + a)^2}{12}\right)$$

$$V = 3850\left(\overline{9.25}^2 - \frac{\overline{76}^2}{12}\right)$$

$V = 3850(85.56 - 40.1) = 175,000$ lbs.

$$v = \frac{V}{4(2d + a)\ jd}$$

$v = 175000/4(76)(.867)(26) = 25.6$ lbs. per sq. in.

This is well within the allowable which by J.C.S. Section 132 $.02f'_c$.

The Moment at the face of the column = $M = \frac{1}{2}w(a + 1.2c)c^2$

$M = \frac{1}{2} \times 3850(2 + 1.2 \times 2.5)\ 6.25 = 60,146.25$ ft. lbs.

$A_s = M/f_s jd = 60146.25 \times 12/18000 \times .867 \times 26 = 1.78$ sq. in.

Use 9 -$\frac{1}{2}$ in. round bars, giving 14 in. bond.

Check Bond Stresses:

$V = \frac{1}{4}w(L^2 - a^2)$

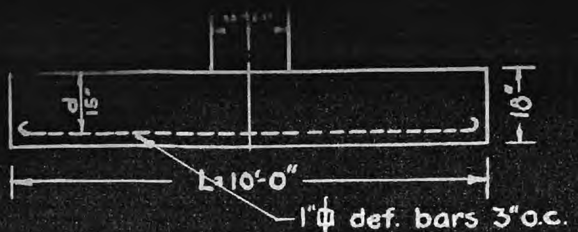
$V = \frac{1}{4} \times 3850(85.56 - 4)$

$V = 962.5(81.56) = 78500$ lb.

$u = V/$ Summation $0\ jd$

$u = 78500/\ .867 \times 26 \times 14 = 25$ lb. per square inch.

This is well within the allowable limit set by J. C. S. Section No. 197.



WALL FOOTING
Fig. Nº 1

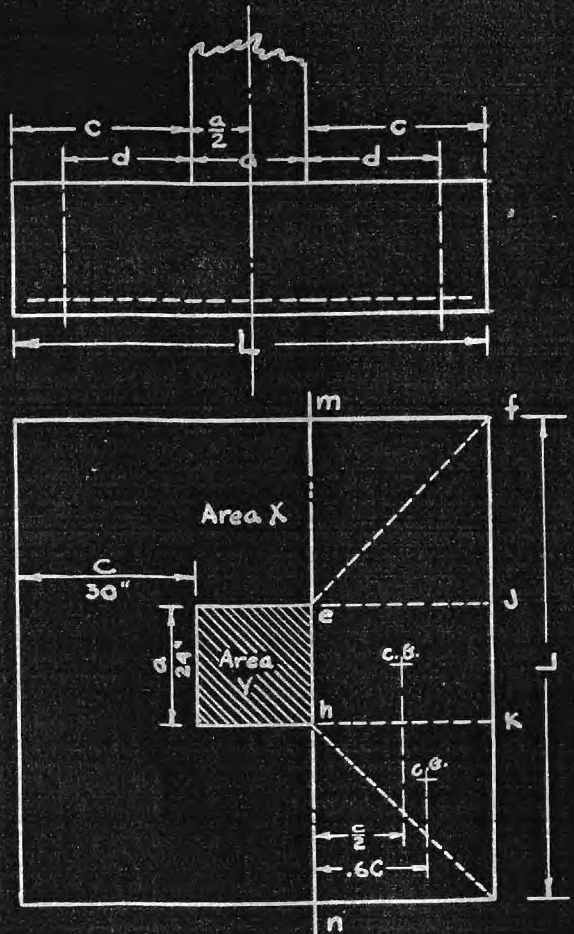


Fig. Nº 2

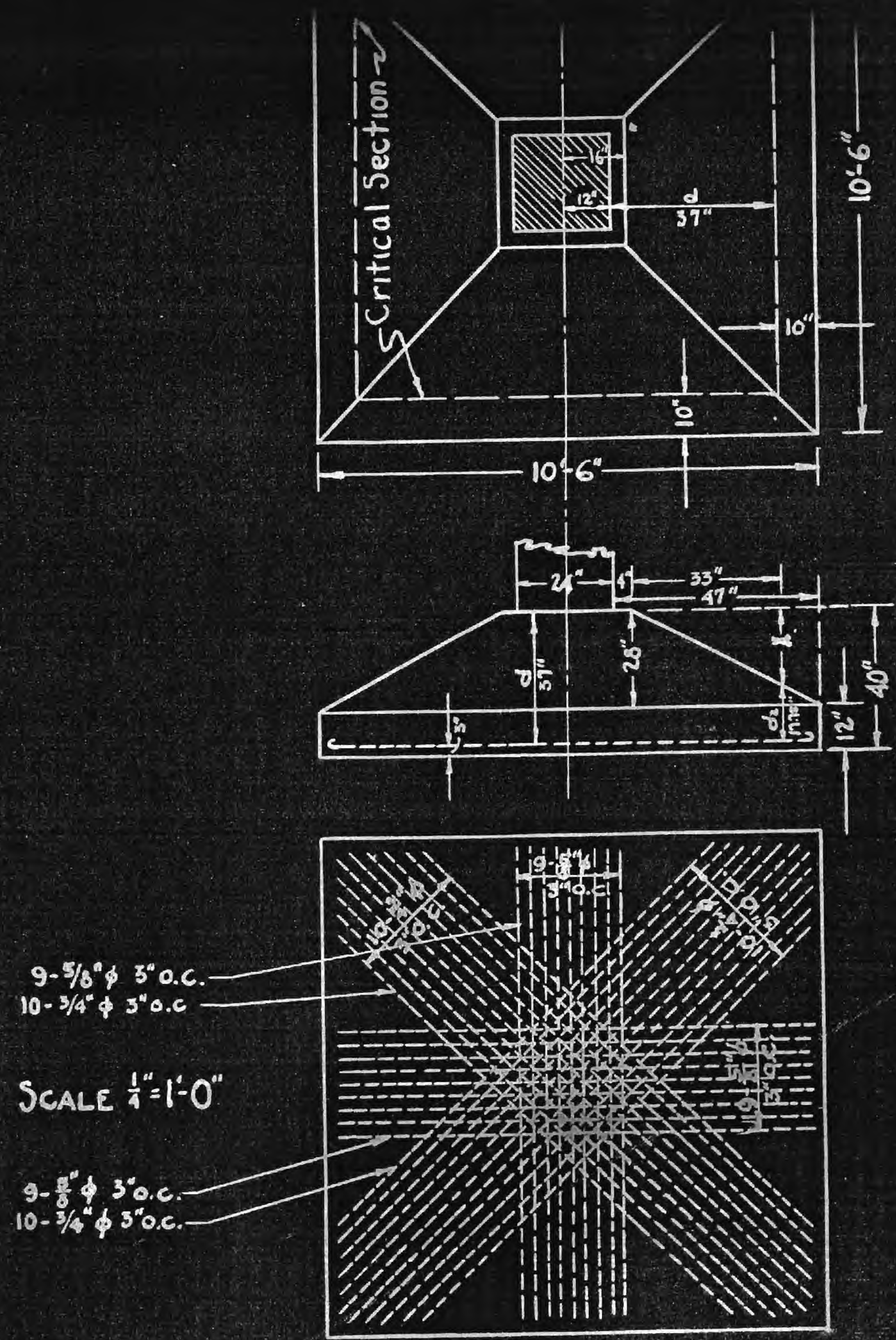


Fig. Nº 3

PLATE Nº 3

Design of a Four-way Reinforced Sloped Footing.

Design a sloped footing, reinforced in four directions, according to the Joint Committee Specifications. The footing supports a column 24 inches square carrying a load of 400,000 pounds. The safe bearing capacity of the soil is 4500 pounds per square foot. Assume a concrete whose f'_c = 2000 lbs. per square inch. For drawings of footing see Plate No. 3, Fig. No. 3.

Assume the weight of the footing equal to 40000 pounds.

The bearing area required = 440000/4500 = 107 sq. ft.

A base 10 ft. 6 in. square furnishing a bearing area of 110.25 square feet is selected. Then, the unit upward pressure, w, = 440000/110.25 = 3995 lbs. per sq. ft.

PUNCHING SHEAR: The total punching shear at the face of the column is = $w(L^2 - a^2)$, and the required depth for punching shear, $d_p = w(L^2 - a^2)/z \times u_p$ u_p = 120 lb. per sq. in. d_p = 4000(110.25 - 4)/4x24x120 = 36.85 in.

Use an effective depth of 37 in. d + d' = 37 + 3 = 40 in.

The upper surface of sloped or stepped footings should project at least 4 to 6 in. beyond each face of the column supported by the footing; therefore the top of this footing is made 32 in. square. The total thickness at the edges is made 12 in.

DIAGONAL TENSION: Investigating diagonal tension along a vertical plane "d" distance or in this case 37 in. from the face of the column, we find the effective depth at this plane equal to: $d_2 = d - X$

33:X = 47:28, 47X = (33)(28), X = 19.25

d - X = 37 - 19.25 = 17.75 in.

And the area outside of this plane is equal to:

$$L^2 - \left(\frac{2d + a}{12}\right)^2 = 110.25 - \left(\frac{2x37 + 24}{12}\right)^2$$

110.25 - 63.68 = 36.57 sq. ft.

$v = V/4b_2jd_2 = 4000 \times 36.57/4 \times 106 \times .867 \times 17.75$

v = 22.4 pounds per sq. in. This is well within the allowable limit. See J. C. S. Section 132.

MOMENT: $M = \frac{1}{2}w(a + 1.2c)c^2$

$M = \frac{1}{2} \times 4000(24/12 + 1.2 \times 51/12)(51/12)^2$

$M = 2000(2 + 5.1)(18.06) = 256{,}452$ ft. lbs.

$A_s = M/f_sjd = 256452 \times 12/18000 \times .867 \times 37 = 5.35$ sq. in.

Since this moment is resisted by two bands of steel, there is one-half or this amount or 5.35/2 = 2.67 sq. in. required in each band. This is satisfactorily furnished by 9-5/8 in. round bars spaced 3 inches on centers. See Table No. 1.

BOND STRESSES: We must check the unit bond stress at the face of the column. U = V/ Summation Zero x j x d

$$u = \frac{1/8\ w(L^2 - a^2)}{\text{Sum. } 0 \times j \times d} = \frac{1/8 \times 4000(110.25 - 4)}{9 \times 1.964 \times .867 \times 37}$$

u = 93 lb. per sq. in.

This is in excess of the allowable value for deformed bars which is 75 pounds per square inch. So we find that bond governs and we must calculate the steel needed to satisfy these requirements.

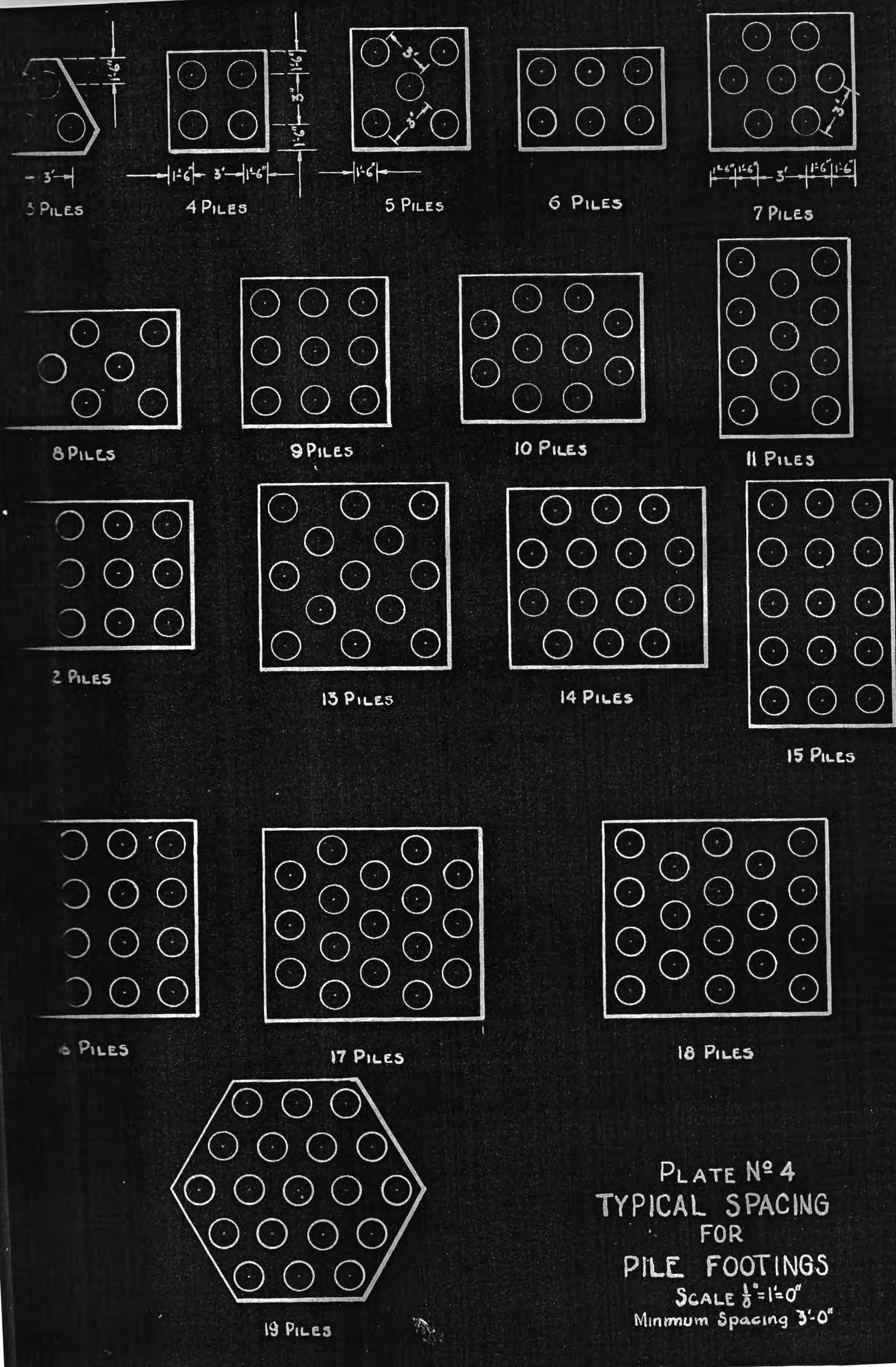
$$\text{Summation Zero} = V/ujd = \frac{1/8(4000)(110.25 - 4)}{75 \times .867 \times 37}$$

Summation Zero = 22 inches.

Use 10-$\frac{3}{4}$ inch round deformed bars spaced 3 inches on centers.

Independent Footing with Piles.
If piles are to carry the column load, the size and shape of the footing will depend upon the spacing of the piles. Plate No. 4 shows typical spacing of different numbers of piles. The minimum spacing should be three foot although, sometimes under very exceptional cases 2 foot 6 inches may be used. The concrete footing capping the piles must be carried down six inches below the tops of the piles. This concrete must not be considered in the strength computations. The bearing power of piles like soil should be determined by a number of tests as described in the Section on Piling.

In the design of footings supported on piling, the upward reaction of the piles should be treated as concentrated loads. In calculating the punching shear, the piles outside of the column or pedestal area need only be considered. The bending moment to be used in calculating the area of the steel is equal to the reactions of the piles multiplied by their distance from the pedestal or column. The reinforcing required by the bending moments should be distributed over the whole effective width of the footing uniformly, as in the case of footings resting on soil. See J. C. S. Sect. 177. The object of the reinforcement is to make the whole footing act as a unit, and such an arrangement as above reduces the number of layers of steel necessary by allowing more room for spacing and also takes care of cross bending. Some designers concentrate the reinforcing over the rows of piling, but this is wrong, because it makes no allowance for cross bending and necessitates unusually close spacing of the steel.


1'-6"
3'
1'-6"
3'
1'-6"
3'
3'
1'-6"
3'
1'-6"
1'-6"
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PILES
4 PILES
5 PILES
6 PILES
7 PILES
8 PILES
9 PILES
10 PILES
11 PILES
PILES
13 PILES
14 PILES
15 PILES
PILES
17 PILES
18 PILES
19 PILES
PLATE Nº 4
TYPICAL SPACING
FOR
PILE FOOTINGS
SCALE 1/8"=1'=0"
Minimum Spacing 3'-0"

FOOTING ON PILES

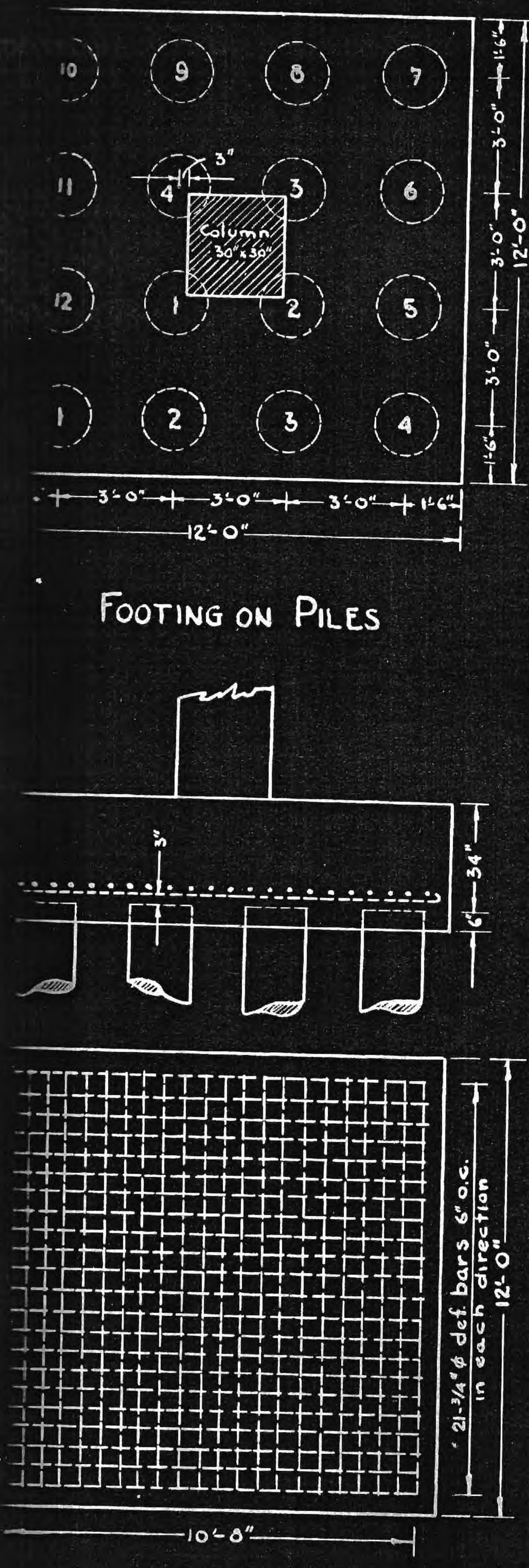


FIG Nº1

BULBS OF PRESSURE

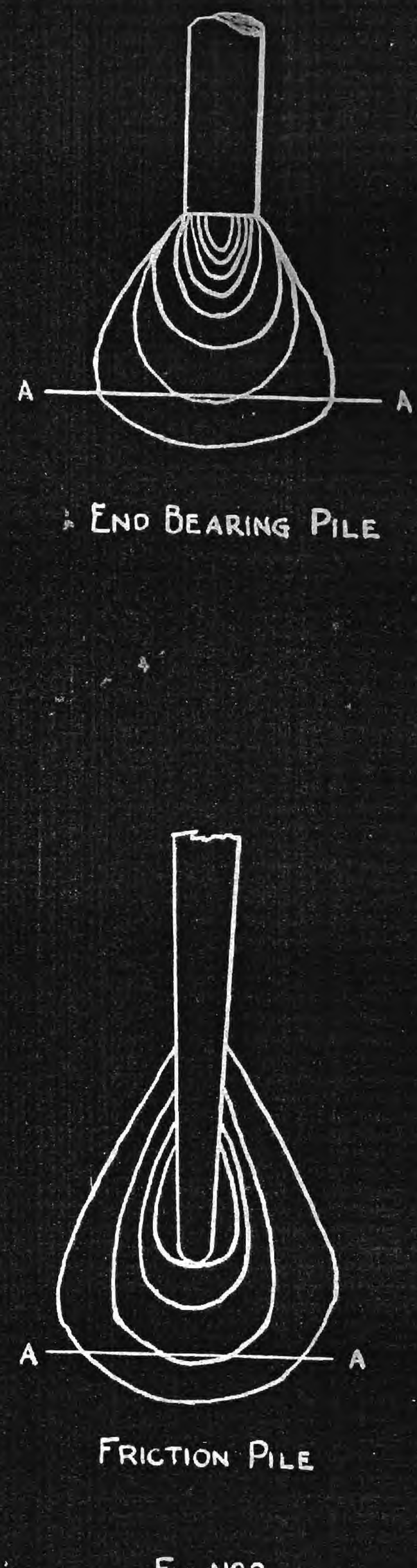


FIG.Nº2

Design of an Independent Footing Supported on Piles.

It has been decided to use a concrete footing supported by piles to support a column 30 inches square carrying a load of 440,000 pounds. The safe sustaining or bearing power of each pile was found by load tests to be 16 tons. For plan of this footing see Plate No. 5 Fig. No. 1. Design the Footing by the Joint Committee Specifications using a concrete f'_c = 2000 lbs. per sq. in.

Assume the weight of the footing = 72,000 pounds.

Total weight = 440000 + 72000 = 512,000 pounds.

The number of piles required = 512,000/32,000 = 16

The load on each pile = 512,000/16 = 31,900 pounds.

In order to keep a minimum spacing of 3 ft. between each pile and to distribute the load equally to the 16 piles, the arrangement shown in Fig. 1, Plate 5 is adopted.

PUNCHING SHEAR: d = Load on Footing/$4a \times u_p$

$d = 440,000/\ 4 \times 30 \times 120 = 30.6$ in. Use 31 inches.

The total thickness of the concrete allowing a 6 inch embedment of the piles in the concrete and 3 inches from the top of the piles to the center of the reinforcing steel gives a total depth of 40 inches.

Check the weight of the footing which equals

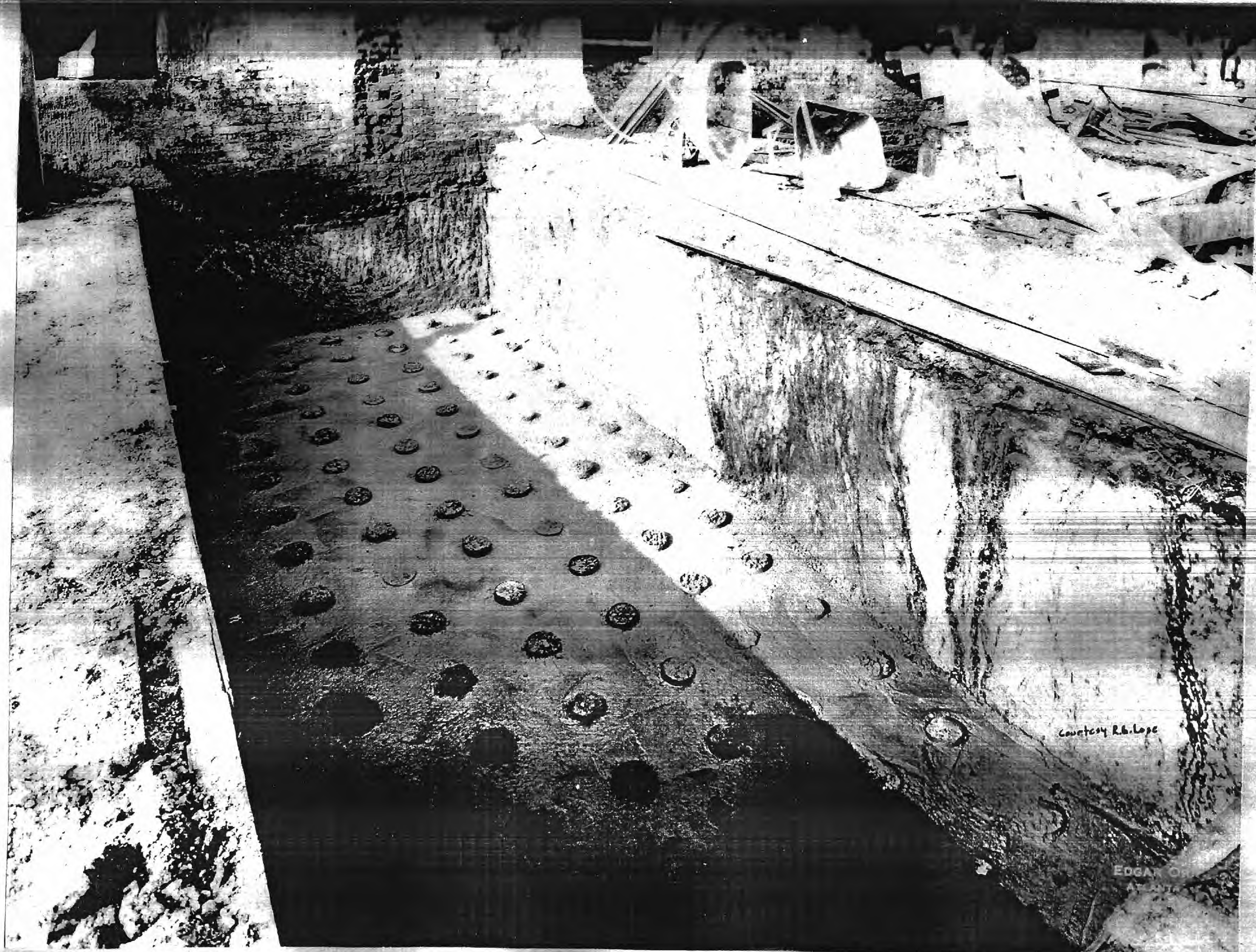
$12 \times 12 \times 40/12 \times 150 = 72,000$ pounds. This checks the assumed weight

Moment: Net load per pile = 31,900 pounds.

The total moment existing around the entire perimeter of the column is: $M = (31,900 \times 3.25 \times 12 + 31,900 \times .24 \times 4)$

$M = 1,242,000 + 31,900 = 1,273,900$ ft. lbs.

Assuming the reinforcing in two bands, the area required in each band is: $A_s = \frac{1}{4}(1,242,000)(12)/18000 \times .867 \times 26 = 9.2$ in^2



Excavation For Footing Showing Pin Piles

Continuous Footing Showing Reinforcing

Combined or Continuous Footings.

It sometimes becomes necessary to combine the footings of two or more columns for one or more of the following reasons: 1. When a wall column is placed at the building line and it is not permissible to project the footing out side of the building line. An independent footing in such a case would be eccentrically loaded. To insure even distribution of the column load on the foundation, the footing of the wall column is combined with the footing of the adjoining interior column.
2. Combined footings are sometimes used for a row of columns when the soil is compressible and it is desired to avoid independent settlement of any one column.
3. Combined footings may also be used where the distance between adjoining columns is so small that it is cheaper to construct a continuous footing than several independfootings. Such may be the case with corridor columns.

General Requirements.

To insure uniform distribution of column loads on the soil, it is absolutely necessary that the center of gravity of the upward reaction of the soil should coincide with the center of gravity of the column loads. If the footing rests directly upon the soil, then this requirement is fulfilled if the center of gravity of the area of the footing coincides with the center of gravity of the column loads. For pile foundations, the center of gravity of the piles and not necessarily that of the footing, must coincide with the center of gravity of the column loads.

Shape of Footing.
If a combined footing is used for two unequal column loads, the base may be made in the shape of a trapezoid, the center of gravity of which coincides with the center of gravity of the column loads, provided the footing rests upon soil. The parallel sides are placed at right angles to the line joining the two column centers, the longer side of the trapezoid being at the heavier column. However, it is usually more desirable to make the footing rectangular in shape, because of its greater simplicity in design and construction. In this case the inequality in column loads may be taken care of by extending the footing the required length beyond the heavier column. With equal column loads, the combined footing should be symmetrical where possible.

Computing the Footing Base.
1. The required area of the footing is calculated from the allowable bearing pressure of the soil or the arrangement of the piles.

2. The center of gravity of the column loads is found.

3. This point is then taken as the center of gravity of the rectangle or trapezoid, depending upon the shape which is selected.

4. The dimensions of the base are then so selected as to give the required area around the above point as its center of gravity.

If the combined footing is for a wall column and an interior column, the footing at the wall column may have to be made flush with the wall. The distance from the center of gravity of the column to the outside face of face of the wall is fixed. This fixes the length of the rectangle, because its total length must be equal to twice this fixed distance, which is from the center of gravity of the column loads to the outside face of the wall. When the length has been determined, the width is found by dividing the rquired area of the base by the length of the rectangle. If a combined footing is for two interior columns, the length of the rectanlge is not fixed and any number of combinations of length and width are possible. The arrangement that has proved most economical is the one where the length of the longer cantilever measured from the outside face of the column, is equal to 0.35 ofthe net span between the columns. After the length is determined, the width is determined from the rquired area.

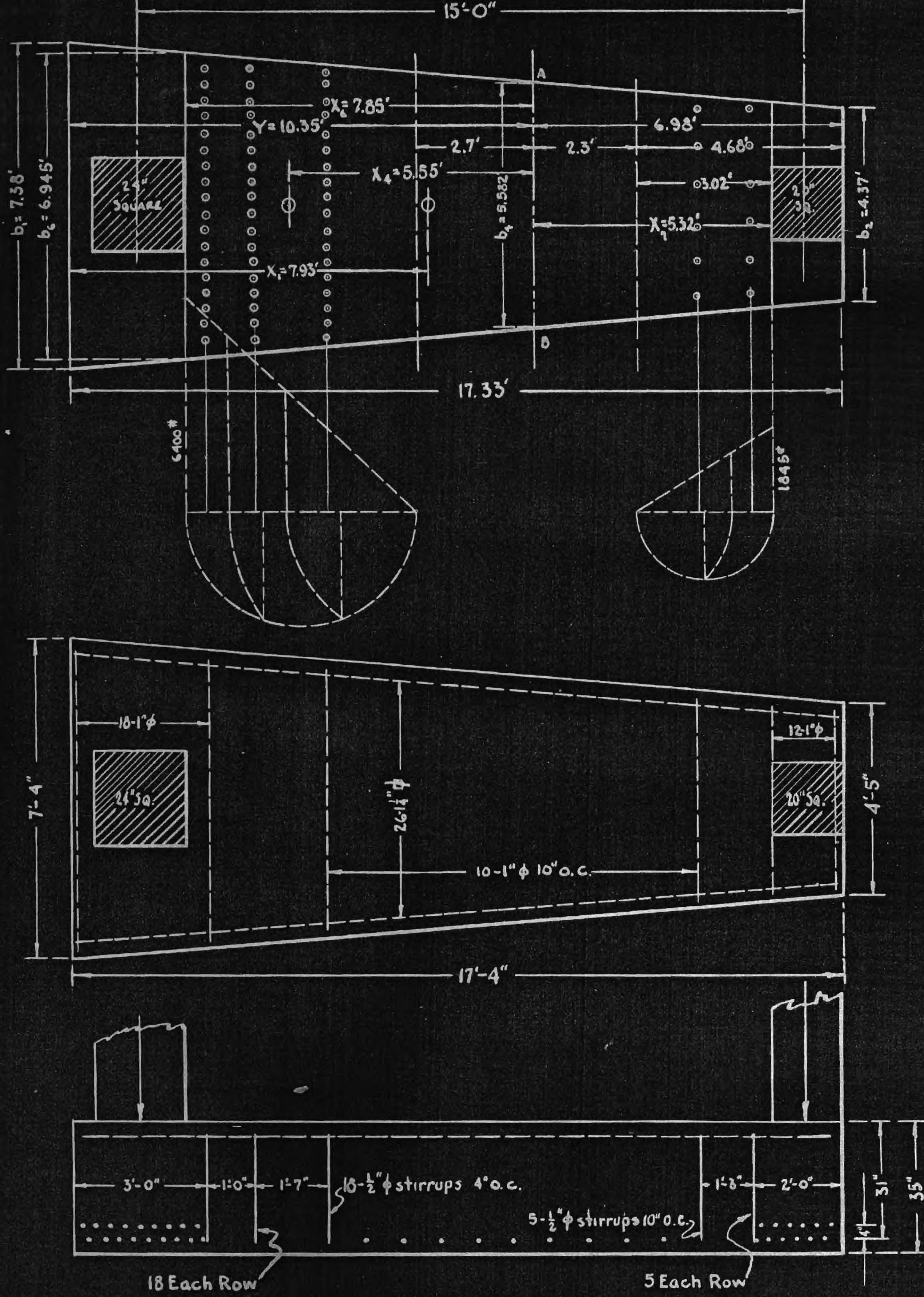
Design Principles.

In most cases the moments and external shear in a combined footing are statically determined. The footing is then designed as any other reinforced concrete footing. The footing may be simply a slab or uniform thickness, however, if the depth required for shear is greater than that required for bending moment and diagonal tension, a pedestal of proper depth and width may be placed over the slab at the column. Sometimes when the shearing stresses permit some of the concrete in tension may be eliminated, using an inverted T-section for the cross section of the footing.

Design of a Trapezoidal Shaped Combined Footing.

Two columns 15 ft. center to center and having cross-

COMBINED TRAPEZOIDAL FOOTING



section areas of 20 in. square and 24 in. square and sustaining 240,000 and 320,000 pounds respectively. The safe allowable bearing power of the soil has been found to be 6000 pounds per square foot. For Plan and Diagram of this footing See Plate No. 6.

Given the above, design a combined footing to support these columns using f'_c = 2000 lb. per sq. in. and $v = 120$ (using web reinforcing), $n = 15$.

Assume the weight of the footing as 500 lbs. per sq. ft.

The bearing area required = 560,000/6000 - 500 = 102 sq . Ft.

The footing will be allowed to project 6 in. beyond the edge of the larger column.

In order to secure uniform pressure on the soil, the center of gravity of the footing must be at a distance from column M of:

240,000 x 15 = 560,000 x X (Taking moments about Col. M)

X = 240,000 x 15/560,000 = 6.43 ft. distance from column M.

6.43 + 1.5 = 7.93 ft. distance from b_1.

Using the formulas involving the area and center of gravity of a trapezoid we find:

Area of the trapezoid = $\frac{1}{2}(b_1 + b_2)L$

$b_1 + b_2 = 2(102.0)/17.33 = 11.75$ ft.

Center of gravity of a trapezoid = $\dfrac{b_1 + 2b_2}{b_1 + b_2}\,\dfrac{L}{3}$

$$\frac{b_1 + 2b_2}{b_1 + b_2}\,\frac{L}{3} = 7.93$$

$$b_1 + 2b_2 = 3(b_1 + b_2)7.93/L$$

$$b_1 + 2b_2 = 3 \times 11.75 \times 7.93/17.33 = 16.12$$

Solving these equations simultaneously:

$b_1 + b_2 = 11.75$

$\underline{b_1 + 2b_2 = 16.12}$

$- b_2 = -4.37 \qquad b_2 = 4.37$ ft.

$b_1 + b_2 = 11.75 \qquad b_1 = 11.75 - 4.37 = 7.38$ ft.

The maximum moment occurs at the point of zero shear. Let the distance of this point from b_1 be called Y, the load on the column which is to the left of the point of zero shear will be called P, and the net upward soil pressure will be called w. Equating upward pressure and downward load:

$$AB = b_2 + \frac{2(b_1 - b_2)}{2L}(L - Y)$$

$$AB = b_2 + (b_1 - b_2)(L - Y)/L$$

$$P = \left[\frac{b_1 + b_2 + (b_1 - b_2)\left(\frac{L - Y}{L}\right)}{2}\right] Yw$$

$$P = \left[b_1Y - \frac{(b_1 - b_2)Y^2}{2L}\right] w$$

Substituting and solving for Y:

$$320{,}000 = [7.38Y - (3.01)Y^2/17.33]5500$$

$$17.33(320{,}000/5500) = 128Y - 301Y^2$$

$$1000 = 128Y - 3.01Y^2$$

$$3.01Y^2 - 128Y + 1000 = 0$$

$Y^2 - 42.5Y + 332 = 0$ Solving by the quadratic formula:

$$Y = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$Y = \frac{+42.5 \pm \sqrt{(42.5)^2 - 4(332)}}{2}$$

$Y = 10.35$ ft.

$$AB = b_4 = b_2 + (L - Y)(b_1 - b_2)/L$$

$b_4 = 4.37 + (17.33 - 10.35)(7.38 - 4.37)/17.33 = 5.582$ ft.

The distance of the center of gravity of the trapezoid bounded by b_1 and AB, from AB is X_4.

$X_4 = h(2b_1 + b_4)/3(b_1 + b_4)$ where h = distance between b_1 and b_4.

$X_4 = 10.35(2 \times 7.38 + 5.582)/3(7.38 + 5.582)$

$X_4 = 5.55$ ft.

The maximum moment is found by concentrating the upward pressure on the trapezoid b_4b_1 at its center of gravity as located above.

$M = P(Y - 1.5) - \frac{X_4(b_4 + b_1)}{2}(wY)$

$M = 2{,}832{,}000 - 2{,}030{,}000 = 802{,}000$ lb. ft.

The depth of footing required to resist this moment is:

$d = \sqrt{M/Kb_4}$ b_4 is in inches and K is from Table II

$d = \sqrt{802{,}000 \times 12/139 \times 5.582 \times 12} \doteq 32$ inches

$d + d' = 32 + 3 = 35$ inches

Some designers use an approximate method in which the moment at the section through the center of gravity of the entire footing is used, thus avoiding the computations for finding the point of zero shear. This gives a bending moment which differs little from the true bending moment.

The depth required for punching shear at the larger column = d_1 W = Bearing power + wt. of footing

$d_1 = P - a_1{}^2(W)/4a_1\ u_p$

$d_1 = 320{,}000 - 4(6000)/4x24x120 = 25.7$ in.

$d_2 = 240{,}000 - (1.66)^2(6000)/4x20x120 = 23.28$ in.

Both of these values are less than the depth required for moment which is used.

The width of the transverse distributing beam under the larger column is taken as 36 in. and that under the smaller as 20 in.

The moment at the edge of the column due to the upward pressure of the soil is:

$\frac{1}{2} \times P/b_1 \times (\frac{b1-2}{2})^2 \times 12$ For Longer Beam.

$$\frac{1}{2} \times 320{,}000/7.38 \times (\frac{7.38 - 2}{2})^2 \times 12 = 1{,}881{,}600 \text{ in. lb.}$$

$$\frac{1}{2} \times 240{,}000/4.37 \times (\frac{4.37 - 1.66}{2})^2 \times 12 = 598{,}000 \text{ in. lb.}$$

for the shorter beam.

The maximum shear at the edge of the column for the longer beam is: $P/b_1 \times (b_1 - 2)/2 = 320{,}000/7.38 \times \frac{7.38 - 2}{2}$

$= 116{,}800$ lbs.

For the shorter beam: $240{,}000/4.37 \times (4.37 - 1.66)/2$

$= 74{,}000$ lbs.

$$d = \sqrt{M/bK} = \sqrt{1{,}881{,}600/36\text{x}138.9} = 19.4 \text{ in.}$$

$$d = V/bvj = 116{,}800/36\text{x}120\text{x}.867 = 29.6 \text{ in.}$$

The depths required are 19.4 in. and 29.6 in. An effective depth of 32 inches is adopted for the entire footing, moment governing. Adding 3 inches for protective coating, the total depth is 35 inches. The weight of the footing is 1 x 1 x 150 x 35/12 = 440 lbs. which checks favorable with the assumed weight of 500 lbs. per square foot. If the depth required for either of the distributing beams had been greatly in excess of that required for the main slab, it would be economical to use the greater depth for the width of the distributing beam only.

In the main slab, the distance from the point of zero shear to the point where the shear reaches the value for plain concrete is determined by solving the equation for the unit shear at any point z feet away from the point of zero shear, substituting for the unit shear v the definite allowable value for plain concrete v; as follows:

$v' = V/b_5 jd$

$b_5 v = b_4 + (b_1 - b_2)/L \times z$ and $V = (b_4 + b_5)zw/2$

Substituting the values for b_5 and V in the equation for v'.

$V = bv'jd \qquad b_5 = b_4 + \frac{b_1 - b_2}{L} \times z, \; b_5 = 5.582 + .1739z$

$bvjd = V$ Substituting the above values in this equation:

$$12(5.582 - .1739z)(40)(.867)(32) = \left(\frac{5.582 + 5.582 - .1739z}{2}\right)5500z$$

$$13300(5.582 - .1739z) = (5.582 - .0869z)5500z$$

$$2.42(5.582 - .1739z) = (5.582 - .0869z)z$$

$$13.51 - .421z = 5.582z - .0869z^2$$

$$.0869z^2 - 6.00z - 13.51 = 0$$

$$z = \sqrt{\frac{-b \pm b^2 - 4ac}{2a}}$$

$$z = \sqrt{\frac{-6 \pm 36 + 4.7}{.1738}} = 2.3 \text{ ft. toward smaller column.}$$

$$12(5.582 + .1739z)(40)(.867)(32) = \left(\frac{5.582 + 5.582 + .1739z}{2}\right)5500z$$

Solving as above z = 2.7 ft. toward larger column.

The width of the footing along the plane b_6 is:

$b_6 = b_1 - (b_1 - b_2)(2.5)/L \quad b_6 = 7.38 - .1739(2.5) = 6.945$ ft.

The shear at $b_6 = V_6 = (b_4 + b_6)(X_6)(w)$

$V_6 = (5.582 + 6.945)/2 \times 7.85 \times 5500$

$V_6 = 271{,}000$ lbs.

The unit shear is: $v = V/bjd$

$v = 271{,}000/6.945 \times 12 \times .867 \times 32 = 117$ lbs. per sq. in.

The total shear to be taken by the vertical stirrups is:

$$\left(\frac{117 - 40}{117}\right)\left(\frac{117}{2}\right)\left(\frac{6.94 + 6.052}{2}\right)(5.15)(144) = 185{,}594 \text{ lbs.}$$

Using ½ inch round bars, the number of single stirrups required is = $185{,}594/18000 \times .196 = 53$

The stirrups are spaced as shown on Plate No. 6.

In order to stress equally all of the web reinforcing, the triangle representing the shear to be resisted by the stirrups is divided into three parts of equal area, and one row of stirrups placed at the center of gravity of each part. The division may be made graphically by dividing the base of the triangle into three equal lengths, constructing a semi-circle on the base as a diameter, erecting perpendiculars from the third points of the base to intersect the circle, and striking arcs of circles from these intersections, using the zero ordinate point of the triangle base as a center. Vertical planes through the points of intersection just described will divide the triangle into three equal parts. The reinforcing steel is placed in rows intersecting the center of gravity of these parts.

The maximum shear in the main slab along the plane whose width is b_7 is found as follows:

$b_7 = b_2 + (b_1 + b_2)/L \times 1.66 = 4.37 + .1739(1.66)$

$b_7 = 4.66$ ft.

$V_7 = (b_4 + b_7)/2 \times X_7 \times w$

$V_7 = (5.582 + 4.66)/2 \times 5.32 \times 5500 = 150,000$ lbs.

The unit shear is = $v = 150,000/4.66 \times 12 \times .867 \times 32$

$v = 72.5$ lbs. per square inch.

The total shear to be taken by the vertical stirrups is:

$$\left(\frac{73 - 40}{73}\right)\left(\frac{73}{2}\right)\left(\frac{4.66 + 5.184}{2}\right)(3.02)(144) = 35,300 \text{ lbs.}$$

Using $\frac{1}{2}$ inch round rods, the number of single stirrups required is: $35,300/18000 \times .196 = 10$ These are placed as before.

The area of longitudinal steel required in the main slab is: $A_s = 802,000/18000 \times .867 \times 32 = 19.3$ sq. in.

Summation Zero = V/ujd = $271,000/75 \times .867 \times 32 = 130$ inches.

Use 26-$1\frac{1}{4}$ square bars spaced on a plane 4 inches below the top of the footing as shown in Plate No. 6.

Steel in the distributing beams:

Longer Beam, $A_s = 1,881,000/18000 \times .867 \times 32 = 3.77$ sq. in.

Summation Zero = $116,800/75 \times .867 \times 32 = 56.2$ inches.

These requirements are met by 18-1 in. round bars placed 4 inches from the bottom.

Shorter Beam: $A_s = 598,000/18000 \times .867 \times 32 = 1.2$ sq. in.

Summation Zero = $74,000/75 \times .867 \times 32 = 35.6$ inches.

These requirements are satisfied by using 12- 1 inch round deformed reinforcing bars.

One inch round reinforcing bars are placed 20 inches on centers between the distributing beams in order to add rigidity to the footing.

Cantilever Footing.

In place of using a combined footing for a wall footing and an interior footing, it may be desirable to build both footings separately and to take care of the eccentricity of the wall footing by connecting it with the adjoining interior footing by means of a strap beam, which may or may not be used to transmit part of the load to the foundation, depending upon the design. The primary function of the strap beam being to resist the bending moment produced by the eccentricity of the wall column load with respect to the wall footing reaction.
This type of footing is particularly useful when a pile foundation is used for both columns, and it is desirable to place the piles in clusters next to the columns; and also when the footings for both columns consist of caisson piles.

Design Principles.
In determining the bending moments and shears, the strap beam may be considered as a balanced double cantilever supported at the center of the wall footing. The short cantilever carries the wall column load, and the long cantilever carries the balancing reaction, which is provided by the interior column with which the long arm is connected. To make the bending moments statically determinate, the strap beam is considered as free to rotate at the ends. With this assumption, from simple mechanics, the bending moment is zero at both ends and increases to a maximum at the point of support. The strap beam should be reinforced at both top and the bottom to take care of

a reversal of stress due to settlement of the footing. Usually about 1/3 of the reinforcing steel placed at the bottom is placed at the tops.

Design of a Strap Connected Cantilever Footing.

For illustrating the fundamental principles involved in the design of a cantilever footing, the front of the building is assumed placed directly on the street line, so that no encroachment beyond the columns is possible. The exterior column footings along that side are therefore eccentric with respect to the loads they support, and must be tied to the nearest interior footings by means of a concrete beam or strap to prevent overturning.

Design of the Interior Column Footing.

The interior column footing will be square, sloped, and reinforced in two directions. See Plate No. 7. Use the Joint Committee Specifications. The safe resisting pressure of the soil is found to be 6000 lbs. per square ft. A concrete with f'_c = 2000 lbs. per sq. inch is used. Diameter of the column = 30 in. Total load = 400,000 lbs. The size of the equivalent square column = 26.59 inches. Assume the weight of the footing as 26,000 pounds.

The bearing area required = 426,000/6000 = 71 sq. ft.

We will use a base 8 ft. 6 in. square which gives a bearing area of 72.25 square feet.

The net upward soil pressure = 400,000/72.25 = 5540 lbs. per square ft.

The punching shear at the perimeter of a 30 in round column = d = V/3.1416xDiameter of column$\times u_p$

$V = 5540(72.25 - 4.9) = 367,000$ lbs.

$d = 367,000/3.1416 \times 30 \times 120 = 32.4$ inches. Use 33 in.

$d + d' = 33 + 3 = 36$ inches.

The top os the footing is made 3 ft. 2 in. square and the total thickness at the edge of the footing is 12 in. Checkimg the weight of the footing we find it to be equal to: $8.5 \times 8.5 + (3.166 + 8.5)/2 \times 2 \times 8.5$

Wt. = 25,711 pounds which is satisfactory.

The effective depth at a vertical plane "d" distance or 33 inches from the face of the round column is $= d_1$.

$d_1 = 33 - 24/29 \times 24 = 13.14$ inches.

The area outside of this plane $= 72.25 - (102/12)^2 = 0$

$v = 0 \times 5540/4 \times 102 \times .867 \times 13.14 = 0$

The bending moment at the edge of the equivalent square column is $= M = \frac{1}{2}w(a + 1.2c)c^2$ Plate No. 3, Fig. 2.

$M = \frac{1}{2} \times 5540(26.59/12 + 1.2 \times 24.4/12)(24.4/12)^2 \times 12$

$M = 635,000$ in.-lbs.

Steel in each band $= A_s = M/f_s jd = 635000/18000 \times .867 \times 33$

$A_s = 1.23$ square inches.

For bond, assuming deformed bars, J.C.S. allows 75 lbs. per square inch. Summation Zero = V/ujd

Summation Zero $= \frac{1}{4} \times 367,000/75 \times .867 \times 33 = 42.8$ in.

This requires 28-$\frac{3}{4}$ in. round deformed bars,hooked at each end,per band.

The effective width for placing each band of steel is = $D + 2d + \frac{1}{2}(L - D - 2d) = 104$ inches. This gives a spacing of about 3 5/8 inches. In this case no steel will be required outside of the effective width. See J.C.S Sect. 177.

CANTILEVER FOOTING

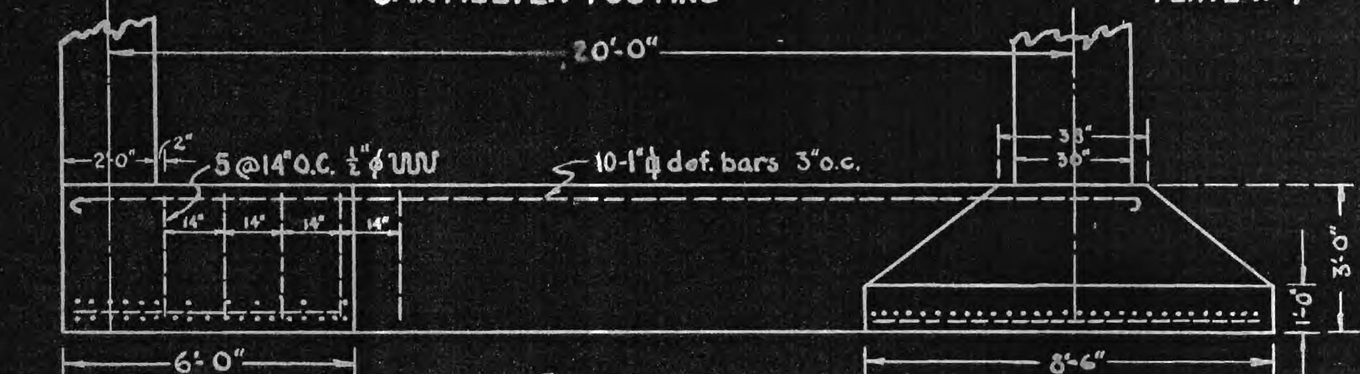


Fig. Nº 1

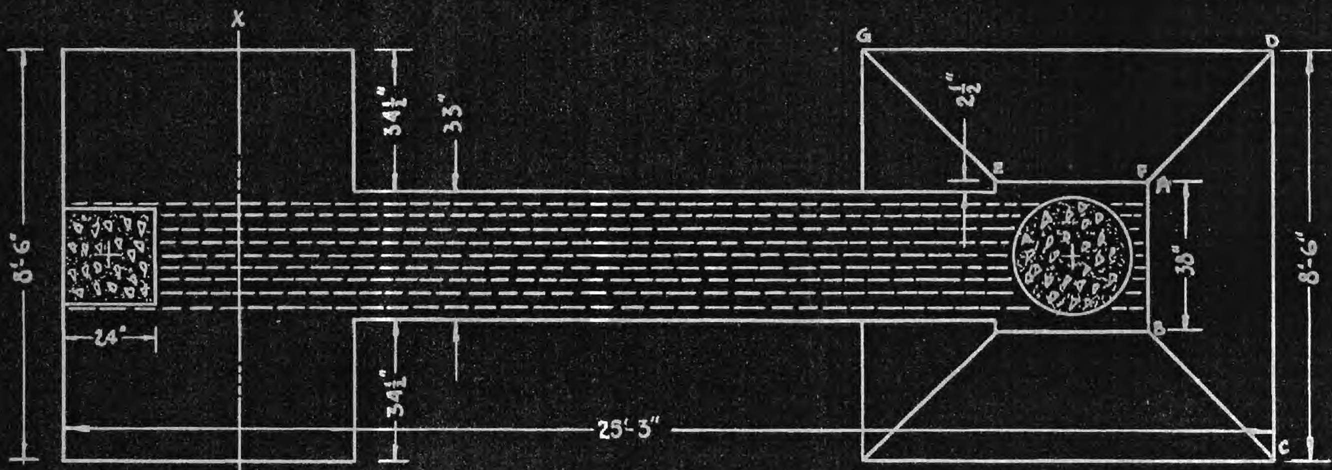
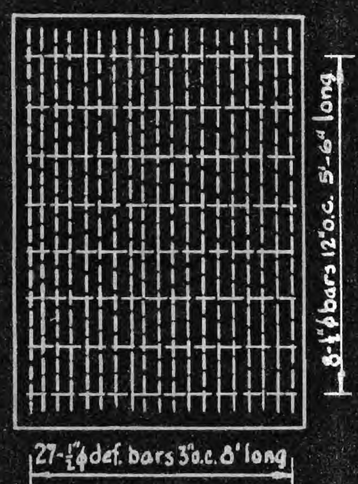
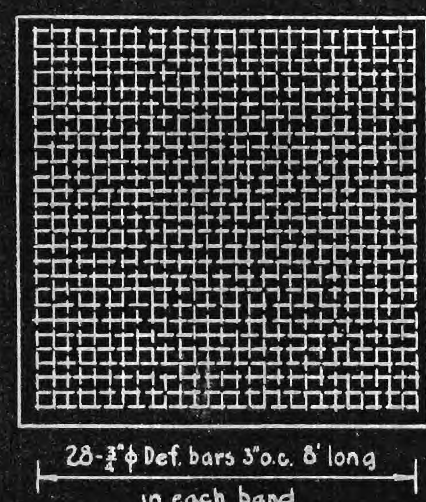
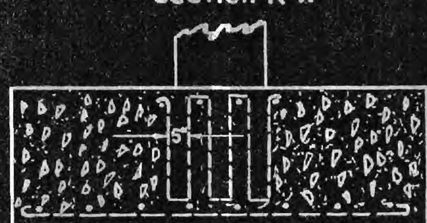


Fig. Nº 2





Section X-X



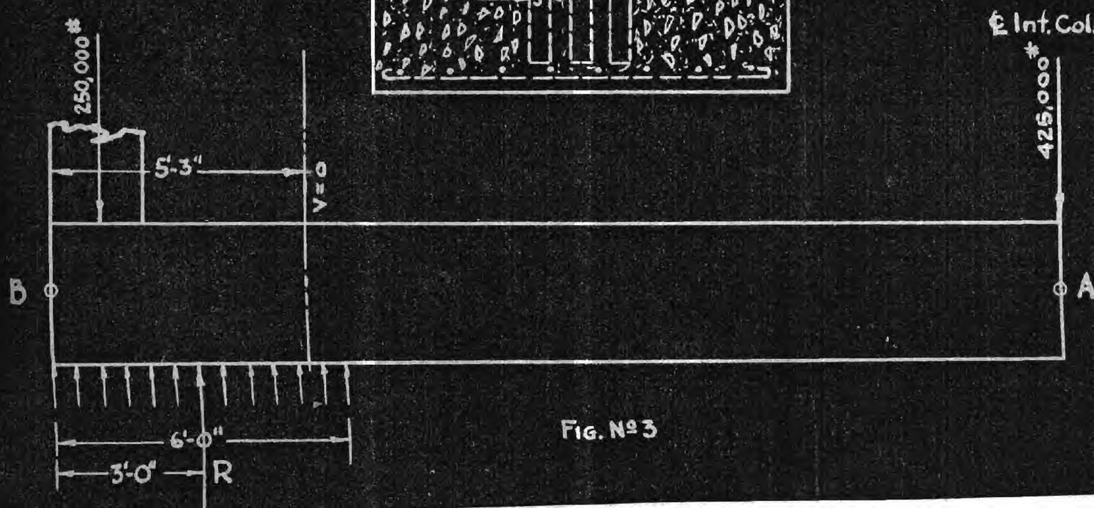


Fig. Nº 3

Design of Exterior Column Footing.

The exterior column footing will consist of a solid block, rectangular footing, reinforced in two directions and connected in the front to the strap or connecting beam.

The column will be 24 in. square and will carry a load of 250,000 lbs. Assume the weight of the footing to be 13,000 lbs.

The bearing area required = 263,000/6,000 = 43.83 sq. ft.

A square footing would be the most economical type to use in supporting a square column, a typical design of which has already been given. However, we will design this footing in the shape of a rectangle for the sake of completeness. The size of the footing selected is 5 ft. 6 in. by 8 ft. 0 in.

The ner upward pressure = 263,000/44 = 5980 lbs. per square ft.

The punching shear at the perimeter of the column is provided for by: d = (44 -4)5980/4x24x120 =20.75 use 21 in.

Allowing 3 in. for a protective coating, d+d'=21+3= 24 in.

The weight of the footing is: 44 x 2 x 150 = 13,200 lbs.

This checks reasonably close the assumed weight of the footing, which was 13,000 lbs.

The shear at a vertical plane "d" distance from the face of the column = $V = (44.0 - \frac{66 \times 66}{144})5980 = 82,200$ lbs.

v = 82,200/4x66x.867x21 =17.1 lbs. per sq. in. which is satisfactory.

The maximum moment occurs along the face of the column parallel to the 5 ft. 6 in. or smaller side of the footing. In rectangular footing design the J. C. S. requires the upward pressure on the trapezoids contributory to the column face, to be concentrated at the center of gravity of the trapezoid, in computing the bending moment.

The total upward pressure on the trapezoid ABCD

$= \frac{5980}{144} \times \frac{24 + 66}{2} \times 36 = 67{,}275$ lbs.

The distance from the face AB to the center of gravity of the trapezoid is: $X_{AB} = h(b_1 + 2b_2)/3(b_1 + b_2)$

$X_{AB} = 3(2 + 2x5.5)/3(2 + 5.5) = 1.735$ ft. $= 20.8$ in.

$M = 67{,}275 \times 20.8 = 1{,}430{,}000$ in.-lbs.

$A_s = 1{,}430{,}000/18{,}000x.867x21 = 4.36$ sq. in.

Summation Zero $= 67{,}275/\ 75x.867x21 = 37$ in.

Bond governs and requires 23-½ in. round deformed bars.

The effective width of the footing for the band of steel is: $24 + 2 \times 21 + \frac{1}{2} \times 30 = 81$ inches. Space 23-½ inch round deformed bars 3½ inches on centers, the for the remaining area use 5-¼ inch round deformed bars 3 in. on centers. See J. C. S. Sect. 177.

The total upward pressure on the trapezoid EFDG

$= \frac{5980}{144} \times \frac{24 + 96}{2} \times 21 = 52{,}430$ lbs.

The distance from the face EF to the center of gravity of the trapezoid is $X_{EF} = h(EF + 2DG)/3(EF + DG)$

$X_{EF} = 21(24 + 192)/3(24 + 96) = 12.6$ inches.

$M = 52{,}430 \times 12.6 = 660{,}000$ in.-lbs.

$A_s = 660{,}000/18000x.867x21 = 2.02$ sq. inches

Summation Zero $= 52430/75x.867x21 = 39$ inches.

Shear governs

We will use 20-$\frac{1}{2}$ inch square deformed bars.

The effective width of this band is:

24 + 2 x 21 + 0 = 66 inches, which is the entire width of the footing. The bars are spaced 3$\frac{1}{4}$ inches on centers.

Design of Connecting Strap or Beam.

The design assumes that all of the loads are resisted directly by the strap, the portion under the exterior column is widened and reinforced at right angles to the strap so as to distribute the pressure from the column and strap over an area sufficient to keep the unit soil pressure below the allowable limit. The total depth of the strap and exterior footing is made the same as that of the interior footing to which they connect. The width of the exterior column footing parallel to the wall is made the same as the corresponding dimension of the interior footing. Note: The exterior footing designed previously under this article was for comparison and to illustrate the design with a round column and is not connected with the strap beam..

The area of the exterior footing required for the load on the column = 250,000/6,000 = 41.6 square feet.

Add approximately 25 percent to provide for the weight of the strap and the footing, and a base of 8 ft. 6 in. by 6 ft. 0 in. is adopted.

Taking the strap out as a free body, the external loads act upon it as shown in Plate No. 7.

The eccentricity of the exterior column load is resisted by a downward pressure P, from the interior column, thus counterbalancing the eccentricity which tends to cause the exterior footing to overturn. The load on the exterior column is considered as uniformly distributed over the column base, and the upward soil reaction, R, is assumed uniformly distributed over the 6 ft. length of the exterior footing.

The total height of the interior footing is 36 inches. The effective depth of the strap or connecting beam is taken as 33 inches, allowing 3 inches protective coating. Assuming the weight of the connecting beam as 1200 lbs. per linear foot, the upward soil pressure, R, is determined by taking moments about point A.

Moments about point A = $-250,000 \times 20 - 1200 \times 21 \times 10.5 + 18R$

$-5,000,000 - 264,600 + 18R = 0$

$R = 292,500$ lbs.

The downward pressure, P, is determined by taking moments about the point B.

$250,000 \times 1 + 1200 \times 21 \times 10.5 + 21P - 292,500 \times 3 = 0$

$P = 17,280$ lbs.

The maximum moment occurs at the point where the shear is zero. Zero shear will occur at some point near the interior edge of the exterior footing. Equating to zero the shear at the point X distance from B.

$-250,000 + 292,500(X)/6 - 1200 X = 0$

$X = 5.25$ ft.

The moment at this point is:

$$M = -250,000(4.23) - 1200 \times \frac{(5.25)^2}{2} + \frac{293,880}{6} \times \frac{(5.25)^2}{2}$$

M =-408,500 lb. ft. or -4,902,000 in.-lbs.

$bd^2 = M/K = -4,902,000/139 = 35,350$

Since d = 33 in. b(1089) = 35,300, b = 35,300/1089

b = 32.5 inches for moment.

The critical section for shear in the beam occurs at the inner face of the exterior footing.

V = -250,000 -1200 x 6 + 292,500

V = 35,300 lbs.

b required for shear = V/vjd = 35,300/40x.867x33

b = 30.8 inches.

We will make the connecting beam or strap 33 in. wide.

The weight is then 36/12 x 33/12 x 1 x 150 = 1218 lbs./ft.

This checks the assumed weight satisfactorily.

The actual weight of the exterior footing, exclusive of the strap, is: 8.5 x 6 x 36 - 6 x 1200 = 16000 lbs.

The soil pressure under the footing is: $\frac{292,500 + 15800}{8.5 \times 6}$

= 6050 pounds per square foot.

This checks the given allowable soil pressure close enough, because in the allowable soil pressure a large factor of safety was used.

A_s required in the strap = 4,902,000/18000x.867x33 = 9.54 Sq. in.

Summation Zero in strap = 35,300/40x.867x33 = 30.4 in.

Use 10-1 inch square deformed bars spaced 3 in. on centers.

The shear "d" distance from the inner face of the exterior column is = V = -250,000 - 1200 x 57/12 +292500/6 x 57/12

V = 92,300 pounds

v = V/bjd = 92,300/33x.867x33 = 98 pounds per square inch.

This is in excess of the allowable unit shear and web reinforcing must be used.

Stirrups are therefore placed from the inner edge of the exterior column to the point where the unit shear = 40 lbs.

per square inch.

$v = V/bjd$

$$40 = \frac{250,000 + 1200X - 292,500(X)/6}{33 \times .867 \times 33}$$

X = 4.45 ft. from the inner edge of the exterior column.

The required spacing of $\frac{1}{2}$ inch round triple looped stirrups at the critical section is:

$$S = \frac{6 \times .1963 \times 18,000 \times .867 \times 33}{92,300 - (40 \times 33 \times .867 \times 33)} = 14.9 \text{ inches.}$$

Maximum spacing is 0.45d = .45 x 33 = 14.85 inches.

We will use 5 triple looped stirrups placed 14 inches on centers, starting at a point 2 inches from the inner face of the exterior column.

The unit shearing stress at the inner edge of the exterior footing is:

$$v = \frac{17,280 + 1200 \times 15}{33 \times .867 \times 33} = 37.4 \text{ lbs. per sq. in.}$$

Therefore, no web reinforcing other than that named above is required.

The unit weight on each of the cantilever portions of the exterior footing, exclusive of its own weight, is:

292,500/8.5x6 = 5730 pounds per square foot.

The maximum moment along the edge of the strap, per foot of width, is = M = 5730 x 2.87 x 1.435 = 23,600 lb. ft.

The area of steel in the exterior footing, perpendicular to the strap, per foot of width is: 23600 x 12 = 283,200

A_s = 283,200/18000x.867x33 =.55 square inches.

The total area for the entire footing is then, .55x6 = 3.30 sq. in. which is furnished by 17-$\frac{1}{2}$ inch round deformed bars, hooked at both ends.

The bond stress on these bars is:

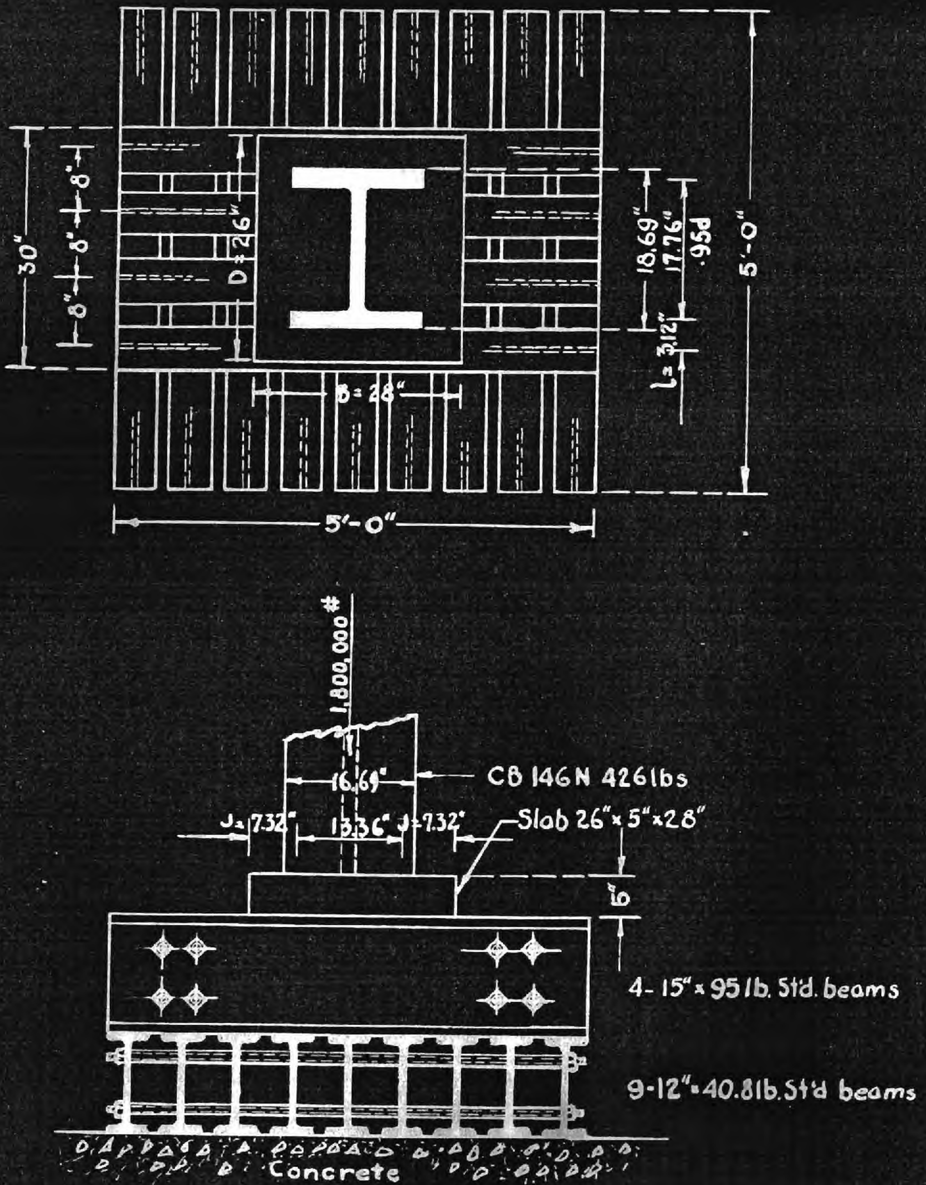
$$u = \frac{5730 \times 6 \times 2.87}{17 \times 1.571 \times .867 \times 33} = 129 \text{ lbs. per sq. in.}$$

Bond governs, so we cannot use 17-$\frac{1}{2}$ inch round bars,

but Summation Zero = $\frac{5730 \times 6 \times 2.87}{75 \times .867 \times 33}$ = 41.4 inches.

We must use 27-$\frac{1}{2}$ inch round deformed bars instead.

Grillage Footing

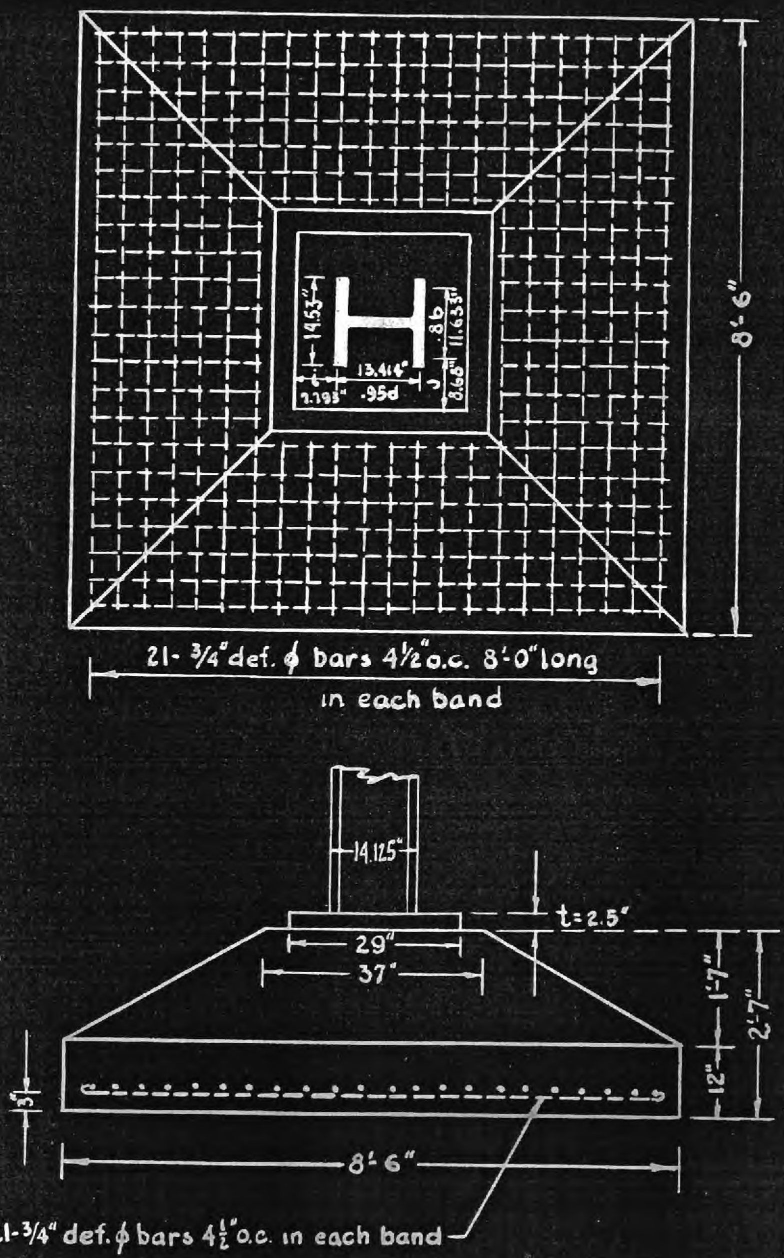


Compressive Strength

500 lbs. per Sq. In.

Scale $\frac{1}{2}$" = 1'-0"

Footing with Metal Bearing Plate



Scale $\frac{3}{8}$" = 1'-0"

Plate No 8

Design of a Footing with a Metal Bearing Plate.

Design a footing to support a column, CB Section 145N weight 95 lbs. per foot, and carrying a total load of 400,000 pounds. (See Carnegie Pocket Companion 1931 and Plate No. 8 in this thesis.)

The safe bearing power of the soil is 6000 lbs. per square foot. Assume the weight of the footing to be 20,000 lbs.

The total bearing area required = 420,000/6000 = 70.0 sq. ft.

The column load P, is assumed to be distributed within a rectangle whose dimensions are o.95d and 0.80b, where d and b in this cases represents the depth and breadth of the column section. The metal bearing plate or slab is considered to act as a cantilever with uniformly distributed load where the span parallel to the web = i= (D - .95d)/2; and perpendicular to the web of the column = j = (B - .8b)/2. P is the total load on the column. D is the smaller side of the slab or metal bearing plate in inches. A is the area of the footing. B is the larger side of the metal bearing plate in inches. p is the unit pressure = P/A.

M = Moment for 1 inch width of metal bearing plate along B or D = $p(i)(i/2) = pi^2/2$ or for the other side $pj^2/2$. Use the value of i or j which is greater than the other.

S = section modulus, for 1 inch width of slab = M/18000

$S = pi^2/36000$ or $pj^2/36000$. Since S = t/6; therefore $t^2 = pi^2/6000$ or $pj^2/6000$.

Required area of the bearing plate = P/Unit compressive stress allowable in concrete footing.

Required area of bearing plate = 400,000/500 = 800 sq. in.

A plate 29 inches square is chosen and furnishes a bearing area of 841 square inches.

Projection i =(D - .95d)/2 = (29 - 13.414)/2 = 7.793 in.

Projection j =(B - .80b)/2 = (29 - 11.635)/2 = 8.682 in.

j^2 = 74.78 Using p = 500 lbs. per sq. in.

$t^2 = pj^2/6000 = (500 \times 74.78)/6000 = 6.23$

$t = (6.23)^{\frac{1}{2}} = 2.5$ inches.

The amount of punching shear at the edge of the bearing plate is: $d = w(L^2 - \frac{DxB}{144})/(4a \times u_p)$

$d = 5800(72.25 - 5.85)/(4 \times 29 \times 120) = 27.6$ inches.

Using an effective depth of 28 inches and a protective coating of 3 inches, we have a total depth of 31 in.

The width of the bearing plate is 29 inches square and the footing top is projected 4 inches beyond this width on all sides, making the top of the footing 37 inches square. The weight of the footing is calculated to be 20,000 lbs. which checks the assumed weight.

Checking diagonal tension at a point 31 inches from the edge of the bearing plate, we find the effective depth at that plane to be = 31 -(31 x 19)/32.5 = 12.9 in. and an area outside of the plane = $72.25 - \frac{(2 \times 31 + 29)^2}{144}$ = 14.79 sq. ft.

$v = (5800 \times 14.79)/(4 \times 91 \times .867 \times 14.123) = 13$ lbs. per sq. in. which is well under the allowable unit stress.

The bending moment at the edge of the bearing plate is =

$M = \frac{1}{2}w(a + 1.2c)c^2 = 5800/2 \times (29/12 + 1.2 \times 36.5/12)\left(\frac{36.5}{12}\right)^2$

M = 161,000 lb. ft. = 1,932,000 in.-lb.

For each band $A_s = 1,932,000/(18000 \times .867 \times 31) = 4.0$ sq. in.

Summation Zero = $\frac{1}{4}$ x 385,000/(75 x .867 x 31) = 46 in.

The above requirements for both bond and moment steel are fulfilled by 20-$\frac{3}{4}$ inch round deformed bars are used and hooked at each end.

The effective width for each band of steel =

$B + 2 \times d + \frac{1}{2}(L - B - 2d) = 29 + 2x31 + 5.5 = 96.5$ in.

This leaves only 5.5 inches and one bar $\frac{3}{4}$ inch round and deformed is used.

Steel Grillage Foundations.

Where it is found to be uneconomical or undesirable to use column base plates, a one or two tier steel grillage may be used to transmit the load to the pier or concrete below the lower tier. The space between the beams forming the grillage are filled with concrete. The beams should be covered on top by not less than 4 inches of concrete and the ends should be enclosed with from 6 to 9 inches of concrete. This concrete is added for protection of the steel and is not considered as adding to the strength of the construction. The steel beams in the grillage must be designed to resist both the maximum bending moment and shearing stresses. Grillage beams were used extensively in the footings of skyscrapers in the early days, but have been largely replaced in this capacity by reinforced concrete construction and are now used largely for distributing very heavy column loads over a foundation structure. Beams in a grillage should never be painted because this weakens the bond with the concrete. In order to prevent the beams from spreading and to insure their acting as a unit, separators made of wrought iron gas pipe with a $\frac{3}{4}$ to 1 inch bolt running through it . These pipes and should be placed at the ends and under all places where concentrated loads occur. If the beams are over 8 in. deep, two rows of separators should be used. The minimum clear distance between flanges of top tier beams = 1 in., lower tier = 2 in. Maximum clear distance for bottom tier = - of the flange width.

Design of a Grillage Foundation.

Design a steel grillage foundation to distribute a load of 1,800,000 pounds, from a CB 146N 14 in. x 16 in. 426 lb. Carnegie Section to a concrete foundation pier, having an allowable bearing capacity of $.25f'_c = .25 \times 2000 =$ 500 lbs. per sq. in. for a 2000 lb. concrete.

The bearing area required = 1,800,000/500 = 3600 sq. in.

60 in. x 60 in. = 3600 sq. in.

Assume a column base plate 26 in. x 28 in.

In determining the thickness of the base plate with respect to the direction of the upper tier, the equation is $t = \sqrt{Pj^2/6000\ BD}$.

In determining the thickness of the base plate with respect to the direction parallel to the bottom tier, it is assumed that the maximum bending moment occurs at the point of column load concentration(.95d or .8b depending on the direction in which the column is turned) and that the reaction from the outside beam in the upper tier is the load, the cantilever span "l" being the distance from the point of maximum moment to this beam and $t = 6Pl/nBf$

Where n = number of beams, P total load on foundation in pounds, and f the maximum allowable bending stress.

18.69 x .95 = 17.76 in. 16.69 x .08 = 13.35 in.

$$\frac{28 - 13.36}{2} = 7.32 \qquad t = \sqrt{\frac{1,800,000 \times (7.32)^2}{6000 \times 26 \times 28}} = 4.7 \text{ in.}$$

$$\frac{24 - 17.76}{2} = 3.12 \qquad t = \sqrt{\frac{6 \times 1,800,000 \times 3.12}{4 \times 28 \times 18,000}} = 4.31 \text{ in.}$$

Use a bearing plate 5 in. thick.

TOP TIER: Use 4 beams 60 in. long and 8 in. on centers.

Section modulus required per beam = $\frac{12P(L - a)}{8fn}$

where a = loaded portion in feet, and L = length of beam in feet.

Section modulus = $\frac{1,800,000 \times (60 - 28)}{8 \times 18000 \times 4}$ 100 $in.^3$

Use 4-15 in. 90 lb. standard beams 4 x 26.12 = 104.48

Buckling stress, $f_b = \frac{P}{n(12a + d/2)t}$ t = thickness beam web.

f_b = 1,800,000/4(27.59 + 7.5)1.068 = 12,000 lbs. per sq. in.

Maximum allowable f_b = 15000 lbs. per sq. in.

Maximum shear f_s at the edge of the slab = $\frac{1,800,000 \times 16}{60 \times 4 \times 15 \times 1.068}$

f_s = 7,500 lbs. per sq. in.

Maximum allowable f_s = 12000 lbs. per sq. in.

BOTTOM TIER: 60in. - 30 in. = 30 in.

Section modulus required per beam = $\frac{1,800,000 \times (60 - 30)}{8 \times 18000 \times 9}$

= 41.7 $in.^3$

Use 9-12 in. 40.8 lb. beams giving a section modulus = 44.8 $in.^3$ each.

Buckling stress $f_b = \frac{P}{n(12a + d/2)t}$

f_b = 1,800,000/(9 x 36 x .46) = 12,100 lbs. per sq. in.

Maximum allowable f_b = 15,000 lbs. per sq. in.

Maximum shear, f_s, at outside edges of upper tier:

= $\frac{1,800,000 \times (60 - 30)/2}{60 \times 9 \times 12 \times .46}$ = 9,050 lbs. per sq. in.

Maximum allowable f_s = 12,000 lbs. per sq. in.

Raft Foundations.

When the allowable pressure on the soil is small, it may be necessary to spread the foundation over the whole area of the building. Such a foundation is called a raft foundation. It is often used in connection with piles, where they are driven in comparatively soft strata and it is desirable to place them as far apart as possible so as not to overload the ground upon which the piles have bearing. The raft foundation is made either of flat slab construction or of beam and slab construction.

Pressure on Raft Foundation.

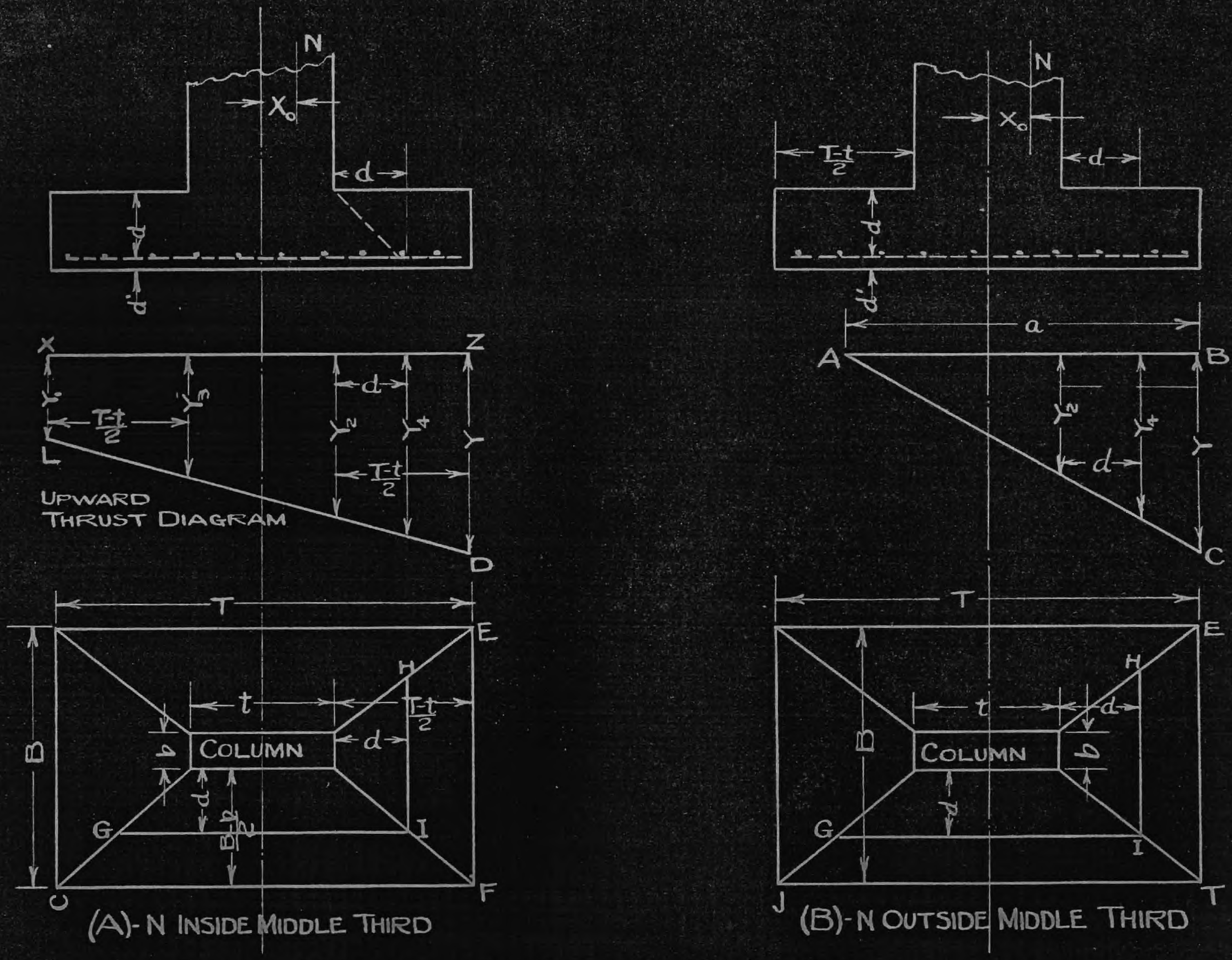
As in the case of independent footings, to prevent unequal settlement, it is necessary to design the footing so that the unit pressure on the soil will be uniform. For uniformly distributed floor loads this is not difficult. If the area of the foundation is equal to the area of the building, then the load on the foundation will be equal to the sum of the loads on all of the floors. With uniformly distributed floor loads, the foundation load will also be uniform.

The difficulty begins when the floor loads are not uniform or when the columns carry additional loads, such as tanks. At wall columns also the load is proportionately larger than at the interior columns, because it consists not only of the floor loading, but also the weight of the wall. To provide for the additional load it is necessary to extend the foundation outside the building lines, when this is not possible uneven pressure on the foundation will result. When this increased pressure does not exceed the maximum allowable pressure, the raft foundation may be used, but the construction between the wall columns and the first row of interior columns should be made strong enough not only to resist the upward pressure, but also to take care of the eccentric moment and even up the settlement of the two columns. For large differences in unit pressures of two columns, the design may sometimes be equalized by using piles under the heavy columns. If a raft foundation is used in connection with piles, the inequality in column loadings may sometimes be taken care of by closer spacing of the piles under the heavier columns.

Types.

In the flat slab floor design type, the pressure on the flat slab acts upward and the column load downward which is the reverse of the floor type loadings. The bending moments would also be opposite. At the columns tensile steel must be at the bottom and at the center it is at the top. To save form work, drop panels may be placed below the slab instead of above it. Instead of using a conical column head, a cylindrical block of concrete equal in diameter to the column head is used as a base, and placed on top of the slab.

The beam and slab construction consists of beams, girders, and slabs. The principles of floor design are applied here also, the loading acting in the opposite direction and the location of the steel will be opposite to that of a floor.



ECCENTRIC COLUMN FOOTINGS

PLATE Nº 9

Eccentric Footings.

When the column load is eccentric and it is not desirable or possible to place the center of gravity of the footing under the theoretical point of application of the column load, then the footing may be designed as an eccentrically loaded footing. There are two classes of eccentric footings, those where the column load falls inside of the middle third, and those where the column load falls outside of the column's middle third. In the first class the upward thrust diagram is a trapezoid and in the second class the upward thrust diagram is a triangle. The design of the two types of eccentric footings follows:

Eccentric Footing, Column Load Outside Middle Third.

t = 1.17 ft. b = 2.00 ft. M from column = 49,000 ft.lbs.

Load = 104,100 lbs.

N = 104,100 + 20 x 2.0 x 1.17 x 150 = 111,100 lbs.

Assumed weight of footing = W = 5,000 lbs.

X_o = eccentricity = 49,000/111,100 = .44 ft.

Assumption is made that the load is inside the middle third, that is, T is greater than 6(.44) = 2.64 ft.

$$BT = .0005\left(\frac{N + W}{L}\right)\left(1 + \frac{6X_o}{T}\right) = .0005\left(\frac{116,100}{5}\right)\left(1 + \frac{2.64}{T}\right)$$

$BT = 11.6 + 30.6/T$, $BT^2 - 11.6T = 30.6$ Make B = 2 + 3 = 5 ft. and T = 1.17 + 3 = 4.17 ft.

$$Y = \frac{N}{BT} + \frac{6NX_o}{BT^2} = \frac{111,000}{5 \times 4.17} + \frac{6 \times 111,000 \times .44}{5 \times (4.17)^2}$$

Y = 9,930 lbs. per sq. ft. maximum upward thrust.

$$Y_1 = \frac{N}{BT} - \frac{6NX_o}{BT^2}$$ = 2270 lbs. per sq. ft. minimum upward thrust.

$Y_2 = Y - \frac{(Y - Y_1)}{2}(1 - t/T) = 9930 - .5(7660)(1 - 1.17/4.17)$

$Y_2 = 7100$ lbs. per sq. ft.

$Y_3 = Y - \frac{(Y - Y_1)}{2}(1 + t/T) = 9930 - 4700 = 5230$ lbs. per sq. ft.

P_1=Punching shear along B $= \frac{(T - t)}{12}\left[B(Y_2-2Y)+b(Y+2Y_2)\right]$

$P_1 = \frac{(4.17 - 1.17)}{12}\left[5(7100 + 2x9930) + 2(9930 + 2x7100)\right]$

$P_1 = 45,765$ lbs.

$M = 3/8(P_1)(T - t) = 3/8(45,765)(4.17 - 1.17) = 51,500$ ft. lbs.

$P_2 = \frac{(B-b)}{12}\left[T(Y_1+Y_2+Y_3) + t(Y_1+Y_2+Y_3)\right]$

$P_2 = \frac{5.12}{12}\ .5 \times 4.17(9930 + 7100 + 4700) + .5x1.17(21730)$

$P_2 = 14,450$ lbs.

$M_{2}v = 3/8P_2(B - b) = 3/8(14450)(3) = 16,250$ ft. lbs.

d_1 B direction $= \sqrt{M_1/Kb} = 51500/139 \times 2 = 13$ in.

d_1 T direction $= \sqrt{M_2/Kt} = 16250/139 \times 1.17 = 10$ in.

d_1 B direction $= P_1/12bv = 45765/12 \times 2 \times 120 = 16$ in.

d_1 T direction $= P_2/12tv = 14450/12 \times 1.17 \times 120 = 8$ in.

Use $d_1 = 16$ inches or 1.33 ft. GI = 3.87 ft. HI = 4.66 ft. from the diagram, See Fig. 9 this thesis.

$Y_4 = Y - (Y - Y_1)\left[\frac{(T - t)/2 - d}{T}\right]$

$Y_4 = 9930 - (9930 - 2270)\left[\frac{(4.17 - 3)/2 - 1.33}{4.17}\right]$

V_1(along HI) = Area ABHI x $.5(Y + Y_4)$

$V_1 = .5(4.66 + 5.00)(.17)(9930 + 8950)(.5) = 7830$ lbs.

V_2(along GI) = Area ACGI x $.5(Y + Y_1)$

$V_2 = .5(3.87 + 4.17)(.17)(9930 + 2270)(.5) = 4150$ lbs.

$v = V_1/(12 \times HI \times jd) = 7830/(12x4.66x.867x16) = 10$ lbs. per sq. in.

$v = V_2/(12xGIxjd) = 4150/(12x3.87x.867x16) = 7$ lbs. per square inch.

A_s(T direction) = $M/(f_s jd)$ = 51500x12/(18000 x .867 x 16) = 2.47 sq. in.

b + 2d = 4.66 Effective width for steel distribution. = = .5(4.66 + 2) + 1.33 = 4.66 ft.

A_s per ft. = 2.47/4.66 = 5.3 sq. in. per ft.

Summation Zero = 7830/(75 x .867 x 16) = 6.5 in.

Summation Zero per ft. = 6.5/4.66 = 1.39 in per ft.

Use 7/8 in round deformed bars 12 inches on centers.

A_s(B direction) = $M_2/(f_s jd)$ = 16250 x 12/(18000x.867x16) = .776 sq. in.

t + 2d = 1.17 + 2.66 = 3.83 ft. T greater than t + 2d

Steel is distributed over a width = .5(4.17+1.17)+1.33 = 4.0 ft.

A_s per ft. = .78/4 = .195 sq. in.

Summation Zero = 4150/(75 x .867 x 16) = 3.95 in.

Summation Zero per foot = 3.95/4 = 1 inch.

Use ½ inch round deformed bars 12 in. on centers.

Eccentric Footing, Column Load Outside Middle Third.

t = 25 in. = 2.08 ft. b = 24 in. = 2.0 ft.

M(col.) = 154,950 lbs. N = 131,600 + (20)(2.08)(2)(150) = 143,600 lbs. Assume W = 5000 lbs.

X_o = 154,950/143,600 = 1.08 ft. Assume N to be outside the middle third. L = bearigg power of the soil = 5 tons per sq. ft.

$B(T/2 - X_o) = (N + W)/(3000L)$

$B(T/2 - 1.08) = (131,600 + 5000)/(3000 \times 5)$

$B(T/2 - 1.08) = 9.06$ Make $B = 4 + 2 = 6$ ft. and $T = 4 + 2.08 = 6.08$ ft.

$a = (.001)N/LB = .001 \times 131,600/(5 \times 6) = 4.37$ ft.

$Y = 2N/(aB) = 2 \times 131,600/(4.37 \times 6) = 10,100$ lbs. per sq. ft.

$$Y_2 = Y(1 - \frac{T-t}{2a}) = 10,100(1 - \frac{6.08 - 2.08}{2 \times 4.37}) = 5560 \text{ lbs.}$$

per sq. ft.

$P_1 = (T - t)/12 \left[B(Y_2 + 2Y) + b(Y + 2Y_2)\right]$

$P_1 = .33 \left[6(5560 + 2 \times 10,100) + 2 \times 5560)\right] = 65,000$ lbs.

$M_1 = 3/8(P_1)(T - t) = 3/8(65,000)(4) = 112,500$ ft. lbs.

$$P_2 = (t + T)/6 \times (B - b)(Y) = (\frac{6.08 + 2.08}{6})(6.2)(10,100)$$

$P_2 = 55,000$ lbs.

$M_2 = 3/8\,(P_2)(B-b) = 3/8(55,000)(4) = 82,500$ ft. lbs.

$d_1 = M_1/Kb = 112,500/139 \times 2 = 20$ in.

$d_1 = P_1/(12vb) = 65,000/(12 \times 120 \times 2) = 22$ in.

$d_2 = M_2/Kt = 82,500/(139 \times 2.08) = 17$ in.

$d_2 = P_2/(12 \times v \times t) = 55000/(12 \times 120 \times 2.08) = 19$ in.

Use $d = 22$ in.

$$Y_4 = Y\left[1 - \frac{.5(T - t) - d}{a}\right] = 9700 \text{ lb. per sq. ft.}$$

HI = 5.87 fn. and GI = 5.9 in. as measured from the diagram plate No. 9.

$V_1 = (5.87 + 6).5(.21)(10,100 + 9700).5 = 12,375$ lbs.

$V_2 = (5.9 + 6).5(.21)(10,100).5 = 6,300$ lbs.

$v = V_1/bjd = 12,375/(5.87 \times .867 \times 22 \times 12) = 9$ lb. per sq. in.

$v = V_2/tjd = 6,300/5.9 \times .867 \times 22 \times 12 = 5$ lb. per sq. in.

Allowable $v = 40$ lb. per sq. in.

A_s(T direction) $= M_1/f_sjd = 112500 \times 12/18000 \times 22 \times .867 = 3.9$ sq. in.

$b + 2d = 2 + 2(1.83) = 5.66$ B is less than $b + 2d$.

The steel is distributed over $.5(6+2) + 2 = 6$ ft.

A_s per ft. $= 3.9/6 = .65$ sq. in. per ft.

Summation Zero $= V_1/ujd = 12375/75x22x.867 = 8.6$ in.

Summation Zero per foot $= 8.6/6 = 1.43$.

Use 1-1 inch round deformed bar 12 in on centers.

A_s(B direction) $= M_2/f_s jd = 82,500 \times 12/18000 \times 22 \times .867$

$A_s = 2.77$ sq. in.

$t + 2d = 2.08 + 2 \times 1.83 = 5.75$

Steel is distributed over $.5(6.08 + 2.08) + 2 = 6.08$ ft.

A_s per ft. $= 2.77/6.08 = .45$ sq. in. per ft.

Summation Zero $= V_2/ujd = 6300/75x.867x22 = 4.72$ in.

Summation Zero per ft. $= 4.73/6.08 = .78$ in.

Use 1 inch round deformed bars 12 inches on centers.

Piles and Pile Driving.

A pile is an element of a structure used to transfer the load on the base of the structure to some strata below it which is capable of sustaining the load.

Types.

Piles may transfer this load to the lower stratum in either or both of two ways, (1) by skin friction along the sides of the piles or (2) by acting as a column, being driven in soft cohesive rock to hard strata. In some cases a large number of piles are driven to better consolidate the soft, non-cohesive ground and improve its bearing power. This type is referred to as pin piles around Atlanta, and is a bout the counterpart of an ordinary fence post. The success of this type pile depends upon the adhesion of the ground to the surface of the pile, namely the the friction between the two.

Wooden Piles.

Wooden piles are usually used where they can be obtained, cut off below the permanent moisture line, and can support the load as in the case of piles acting as columns. Yellow pine and cypress in addition to their availability in the market, are the most suitable woods for pile foundations, in that they are straight, well bodied, free from large top growth, and may be obtained in long lengths. Wooden piles are usually about 12 to 20 inches in length at the butt and about 6 to 12 inches at the point. Wooden piles must be practically straight. A good criterion of straightness is a line drawn from the center of the butt to the center of the point shall be within the body of the pile.

Treating Wood Piles.
If piling is to be of wood and exposed above the line of permanent moisture or to marine borers such as the toredo, (toredo navalis), the limnoria(Limnoria Terebrans), or the Lycoris Fucata, concrete piling is favored. However, due to economic considerations wood piling is sometimes used in the above cases. Should it be used, it should be treated with from 20 to 30 pounds of creosote per cubic foot of timber. Zinc Chloride is not good for piling exposed in this manner. Timber for piles should be cut in the fall of the year when the trees are pracically dormant.

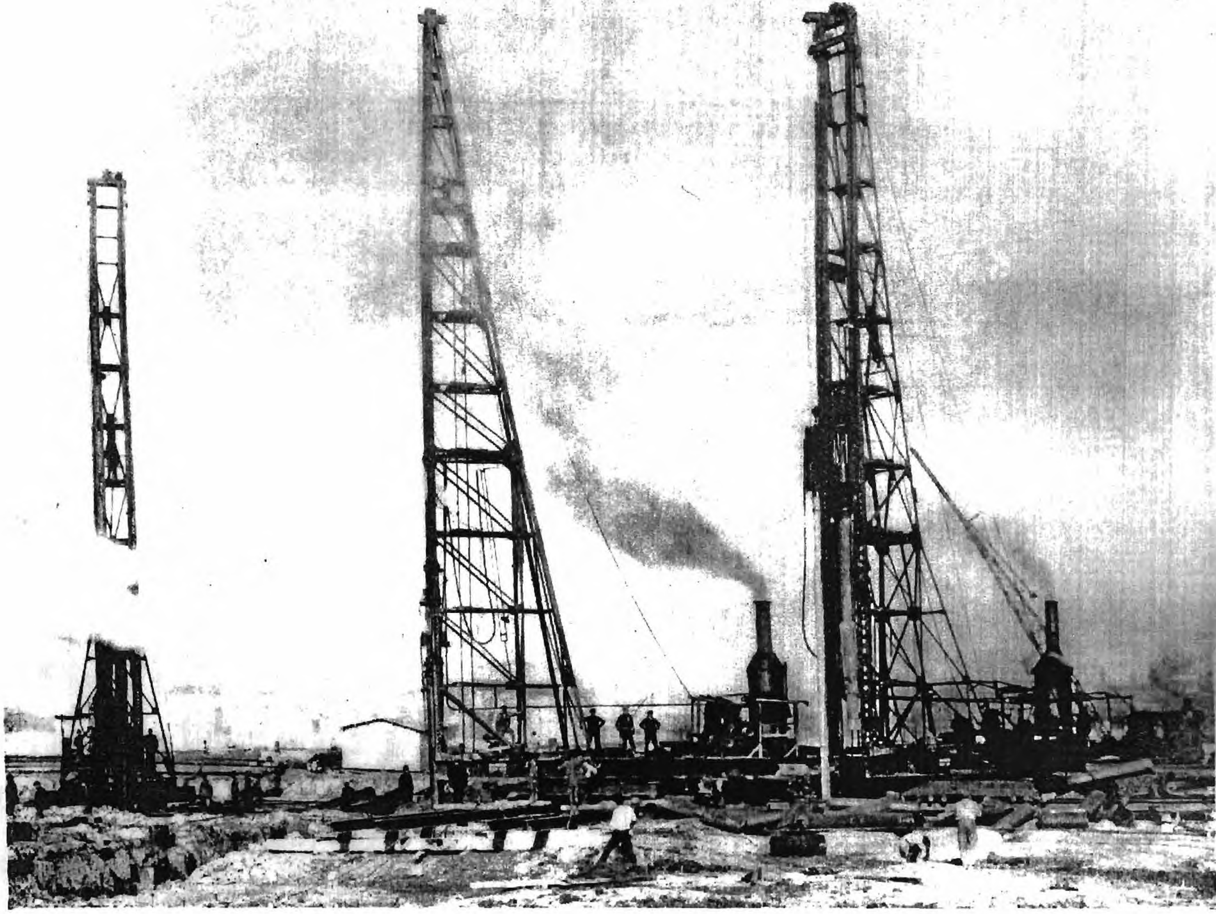
Marine Borers.
I believe that something should be said here about the nature of marine borers, because they are dangerously destructive to wood construction in the waters which they infest.
The Toredo Navalis, or ship worm, which is a species of the mollusk is most destructive. It deposits its eggs on wood construction immersed in water. From these eggs develpp worms whose heads are equipped with a shell like instrument resembling an auger with which it bores its way into the timber parallel to the grain. As it progresses, it lines the tunnel with a calcareous deposit. The toredo continually increases in size as its boring progresses, and specimens as large as ½ inch in diameter and 6 feet in length have been found. The toredo makes his habitat in salt waters of the warmer climates and

is most active along calcareous shores.

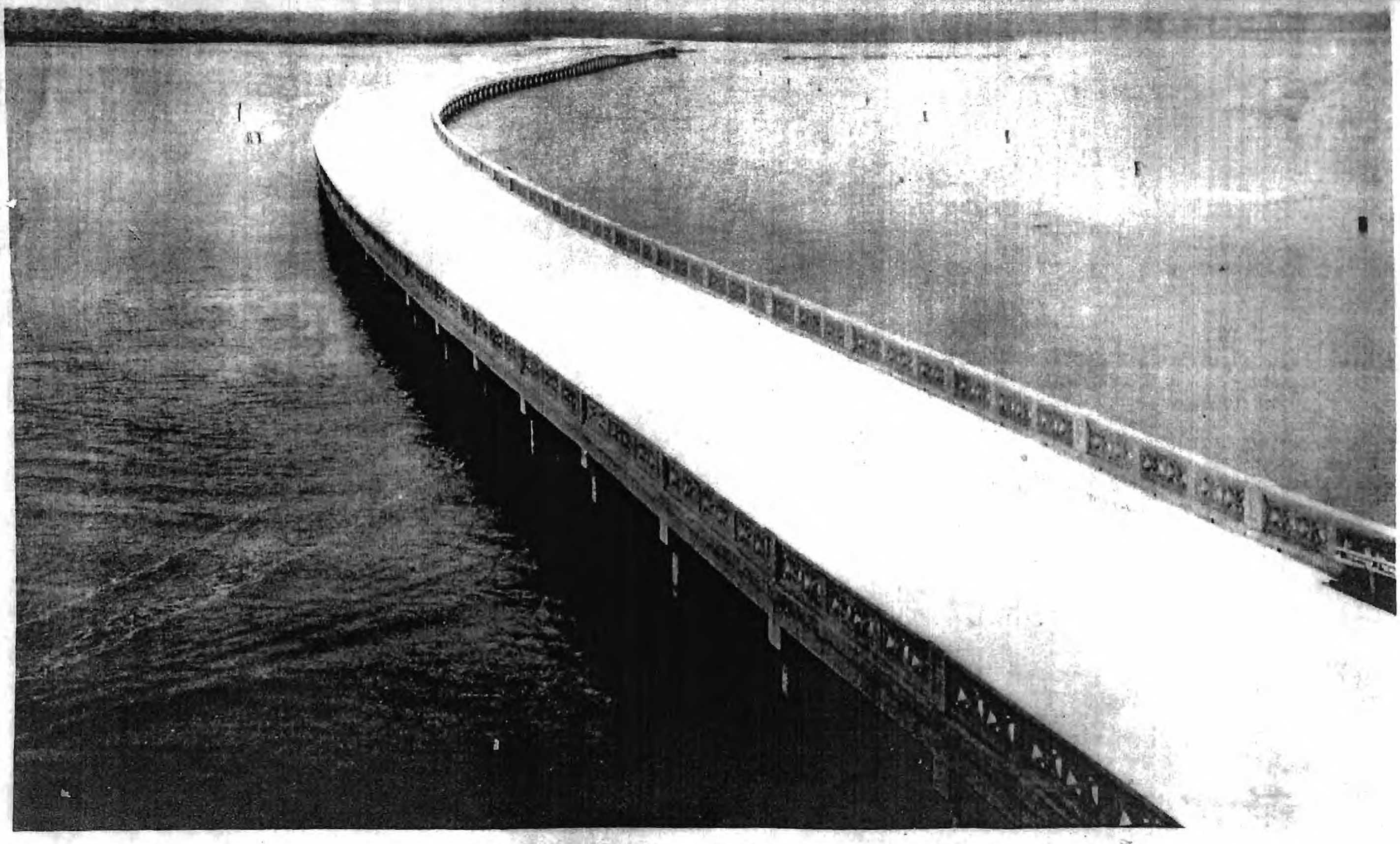
The limnoria or gribble is a crustacean about the size of a grain of rice and resembles the wood louse. It can swim, crawl, and jump and requires both water and air for existence. Unlike the toredo, it does not bore through the wood, but devours it to a depth of about ½ inch by means of claws or mandibles. The surface of the wood becomes undermined and is washed away. The limnoria devours wood in this manner at the rate of one to 3 inches per annum. As many as 20,000 have been counted on one square foot of timber. It has been found both in warm and cold waters and prefers siliceous shores.

The Lycoris Fucata is a little worm similar to the centipede, having many legs. In inhabits the mud and climbs up piles infested with the toredo, destroys him, enlarges the entrance to his burrow and lives in it.

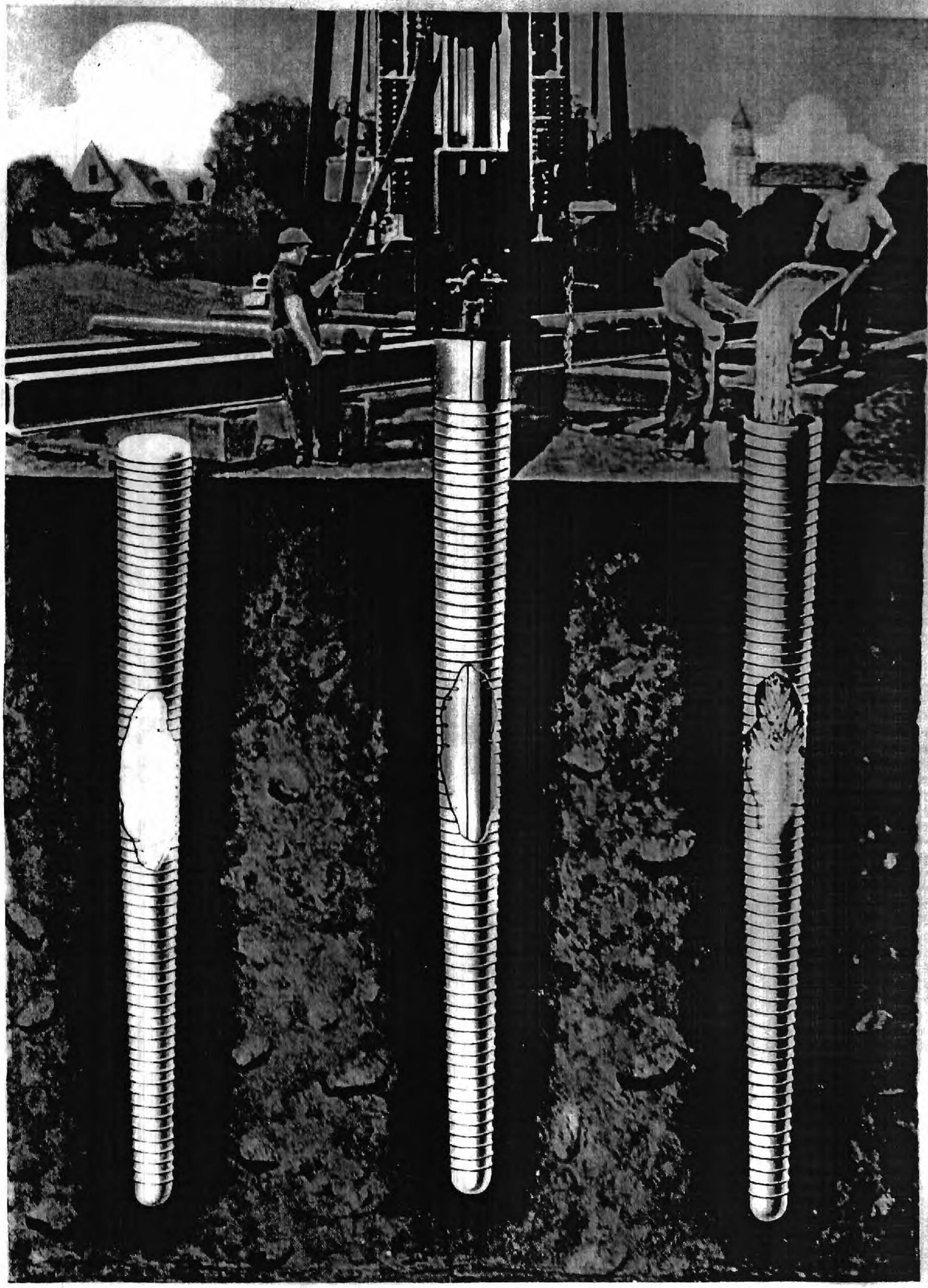
Many protective measures against these marine borers have been tried, none proving entirely satisfactory. Creosoted timber is much more resistant to them than untreated timber, and is more effective in cold waters. However, if the timber checks or cracks after being treated, it will afford entranced to the untreated center which will be quickly attacked. Protective coatings such as asphalt, sheet copper, and burnt terra cotta pipe are expensive and have never proved effective over a reasonable length of time. For the above reasons where the work is to be permanent, concrete piling will prove far more economical and safer than wood piling.



A group of three 85' all-steel drivers operating at the Ford Rouge Plant, and placing composite piles to a length of 80 to 100 feet below ground surface.



Boca Ciega Causeway, St. Petersburg, Florida.



STANDARD RAYMOND CONCRETE PILES

Concrete Piles.

Concrete Piles are usually divided into two classes, namely, those cast in place and those precast. Concrete piles cast in place may be either reinforced or not, but precast concrete piles are always reinforced to take care of the stress developed in handling and driving. The precast type is molded in a form and after curing and hardening is handled and driven like a wooden pile. The foot of the pile is usally supplied with a metallic cap and extra reinforcing steel for protection during driving. All reinforcing steel should have a protective coating of at least 3 inches and the piles should be cast or molded at least 30 days before driving.

Cast in place concrete piles are practically all patented. There are two groups of cast in place piles, namely, piles in which a permanent shell is left in the gound to protect the green concrete from the action of the surrounding water and soil and to prevent its distortion from the natural pressure of the surrounding earth and that caused by driving adjacent piles; and secondly, piles in which no permanent protection remains around the concrete after it has been placed in the ground.

The Raymond pile is the best known of the first class, that is those cast in place piles where a protective coating is left. It is formed by driving a steel shell into the ground on a mandrel that can be collapsed and withdrawn. Then the shell is filled with concrete, which may or may not be reinforced.

The Simplex pile is made by driving down a closed steel pipe and withdrawing it while concrete is being forced out at the bottom. This type of pile is best adapted to stiff, non-water bearing soils.

The MacArthur or Pedestal pile is driven in much the same way as the Simplex pile, but has a spread footing which is obtained by forcing the concrete out at the bottom of the shaft or pile by compressing the adjacent soil It does not have a permanent shell.

The Pretest piles are best adapted to the underpinning of foundations already constructed and for this reason is not discussed here.

If cast in place piles are being driven which have no permanent shell, they should not be driven closer than 5 to 10 feet apart within 7 days after the ones driven preceding them have been allowed to set.

Composite piles of concrete and wood have been used successfully in water bearing soils where long lengths are needed. A long wooden pile is first driven to such a depth that its top will be below the line of permanent moisture. A concrete pile is then doweled on top of the wooden pile and driven, the concrete extending from the water level or permanent moisture line to the desired height. The Raymond Composite Pile is much used in this class.

Steel Cylinder Foundations.

This type of foundation piling depends to a large extent upon the walls of the steel cylinder, which are usually a minimum of 3/8 in. thick, for strength. These cylinders are driven by the use of a pile driver to solid rock and then cleaned out, cut to elevation with an acetylene torch and filled with concrete.

Pile Driving.

The driving of piles for a foundation is almost as important as the design of the foundation itself and should be supervised by experienced men. Piles should be straight, because inclined piles are very inefficient in carrying vertical loads. The depth to rock should be determined by borings inorder to prevent ordering piles too long or overdriving them. If possible all the piles in the same group should be driven to the same depth. Piles may be driven by either a drop hammer or a steam hammer. The required weight of a drop hammer depends upon the size of the pile and the nature of the material into which it is being driven. The weight of hammers used for wooden piles is from 2000 to 5000 pounds and the fall about 20 feet. Drop hammers are recommended for sheet pile work as better control of the blows is obtained. A pile ring or protective cap should be used while driving wooden and concrete piles. Dropshammers are usually used on marine work because they are lighter than steam hammers. The steam hammer which has as an internal part a steam cylinder which raises the hammer a short distance and drops it automatically in rapid succession is rapidly replacing the drop hammer for general use. There are two types of steam hammers those which are single and those which are double acting. The double acting pile hammer not only lifts the ram by steam as in the case of the single acting hammer, but also uses the steam pressure to drive the ram down.

The advantages of the steam hammer over the drop hammer are:

1. The pile is kept in position and guided better.

2. Brooming and splitting of the head is less liable to occur.

3. Driving is equally effective for any position in the leads.

4. Piles may be driven several feet below the leads without the use of a follower.

5. Rapidity of action keeps the pile in motion and facilitates driving. It takes from 1/3 to 2/3 the time to drive a pile with a steam hammer as with a drop hammer.

The modern steam hammer equipped pile driver can drive about 40 piles per eight hour day under average conditions. All piles should be driven practically to the point of refusal. Where the piles are being driven through non-cohesive soils such as sand, the aid of a water jet placed at the tip of the pile may be used. Along the seashore piles may be jetted in placed entirely. The pressure on the jet should be such as to keep a steady flow of water from under the pile to the top of the ground along the sides of the pile. The cost of wooden piles is about 20 to 40 cents per linear foot depending upon the size and the location. Concrete piles cost about two dollars per linear foot in place.

Spacing of Piles.

Piles are usually spaced 3 foot center to center with 2 foot 6 inches center to center as an absolute minimum. Actually the spacing of piles depends to a large extent upon the earth into which they are driven. Precaution must be taken that they are not driven so close together that the stress transmitted by the pile to the soil will exceed the bearing value of the soil. They should not be driven so close together that the stressed areas of soil surrounding any group of piles overlap.

Bearing Capacity.

For convenience in calculating the bearing capacity, piles are divided into two classes which are namely: (1) where the lower end rests upon a hard stratum and receives practically no support from the surrounding earth. In this case the pile should be designed as a column and its bearing capacity will depend more than any other way on its strength as a column. I would recommend that in no case should the bearing capacity of a pile exceed 25 tons. (2) Those which transmit the load largely by friction of its sides with the surrounding earth. This condition unfortunately is more often encountered than Case No. 1.

It would be well to look at the pressure bulbs for each of these types of piling. The end bearing pile forms the bulb of pressure entirely below its point. See Plate No. 5, Fig. 2. The pressure lines of the pressure bulb of the friction pile extend up along the sides of the pile, as shown by the diagram. The plan views of both the bulbs of pressure of these two types are very much the same, and their bearing power depends to a large extent upon their bearing power at section A-A. The size of these bulbs of pressure depends upon the physical characteristics of of the soil in which they are formed. The friction of the pile with the soil can only aid in forming the bulb up to the maximum size. It is therefore apparent that the most economical foundation is the one in which the piles are just sufficient and so spaced as to produce bulbs at the plane A-A which are tangent to each other, and in this way utilizing practically the entire strata at A-A. Using more piles merely cause an overlapping of the stressed zones and in most cases are merely a worthless waste of money. Failures of pile foundations are in most cases caused by the underlying strata at A-A or below being overstressed.

Piles are in many cases wastefully used and add absolutely nothing to the foundation values. This is especially the cases where the material below the depth of the plane A-A is very poor and no better than that at the base of the footing. In a certain location in Texas a building was built in a locality where piles had been almost invariably used. According to the practice prevalent in the design of the immediately surrounding buildings, about 1600 piles would have been necessary in the foundation structure. No piles were used. The building was placed on a reinforced concrete raft foundation. The building settle just about the same amount as the surrounding buildings which had been placed upon piles. Although we have seen it may be erroneous at times one of the best means of determining whether to use piles or not, is by observation of the surrounding structures and careful study of the results obtained.

Dr. Charles Terzaghi in his discussion, "The Science of Foudations, Present and Future;" in Vol. 93, Transactions of the Am. Soc. C. E., divides pile foundations into four classes, namely:

(a) Pile foundations in deep soft mud or poor clay deposits.

(b) Pile foundations in deep,loose sand and clay deposits.

(c) Pile foundations transmitting the load of the super structure through soft strata to more solid strata.

(d) Pile foundations on permanent sedimentary strata, containing at a greater depth, spots of sand or clay.

In Class (a) the pile transmits its load to the soft mud below it, and the action of the piles eliminates the setlement of the structure which would be due to the consoldation of the soil above the pressure area of the piles.

Class (b). This class includes foundations on loose sand. Because of the fact that the weight of the upper strata consolidates to a certain extent the soil it rests upon, the compressibility of the soil decreases rapidly with the depth,to a depth of about 12 feet, and becomes nearly constant at greater depths. Because of these facts, short piles may be used here effectively.

Class (c) This is the case where the piles transmit the load through soft strata to firmer strata below. This is a case where pile driving is very effective, the driving of the piles tending to consolidate the soft upper strata and where this cannot be done, and where the soil is practically incompressible as in the case of plastic soils there is a upward displacement equal to the volume of the piles.

Class (d) This is where pile foundations are placed on permeable, sedimentary strata, interspersed with deposits of soft, plastic clay or mud at a much greater depth. This type of soil causes much trouble, and many large settlements of pile foundations have taken place because of it. These have been due to two causes (1) the consolidation of the deposits due to its drainage by the squeezing out of the excess water; (2) the displacement or motion of the whole plastic deposit. The volume of earth subject to settlement is usually contained in the bulb of pressure which extends to a depth of about one and one-half times the width of the building.

Test Piles.

Where test borings have not been made and it is desirable to know the length of the piles to be used, test piles may be driven. In practically all cases, except where the pile has bearing on ledge rock, the static load test should be used for determining the bearing power of the piles. After the piles have been allowed to stand 24 hours after driving, a platform is built on one to three adjacent piles and loaded by increments until the total load on each pile is from 20 to 30 tons. After the load per pile has reached 20 to 30 tons, the piles are allowed to stand loaded for two or three days, data being taken on the progressive settlement. The load on the piles is then further increased until the limit is reached. The allowable load per pile should not exceed one-half the ultimate load and the writer believes that it should never exceed 25 tons per pile, A record is also made of the-------------------------

recovery of the pile when the load is removed. Tests, if made on a few separated piles not in a cluster are often very misleading as to what a pile in a cluster might bear, except in the case where the piles are driven to rock through very soft strata. In such cases as where the piles are driven in spongy clay ground, where short piles are driven into ground which is very uniform for the length of the pile, they are very misleading. In driving piles in a cluster, acute notice should be kept for shifting ground levels due to earthldisplacements. Driving of piles in clusters in clay soil, have been known to cause the ground level to rise as much as 4 feet in a cluster of piles being driven 3 foot on centers. Accurate records should be kept of the movement of the ground level and studies made to determine the best possible spacing for that particular cluster of piles in that particular soil.

Pile Driving Formulae.

The use of pile driving formulas, because of their in herent inaccuracies are never to be encouraged. One of the reasons for the failure of pile driving formulas to forecast the true bearing power of the piles is the fact that the dynamic and static resistance of the piles may be widely different. Many formulas make no no attempt at all and the best ones only a weak and vague attempt to give suitable constants for piles driven with steam instead of drop hammers.

Deep Foundations on Land.

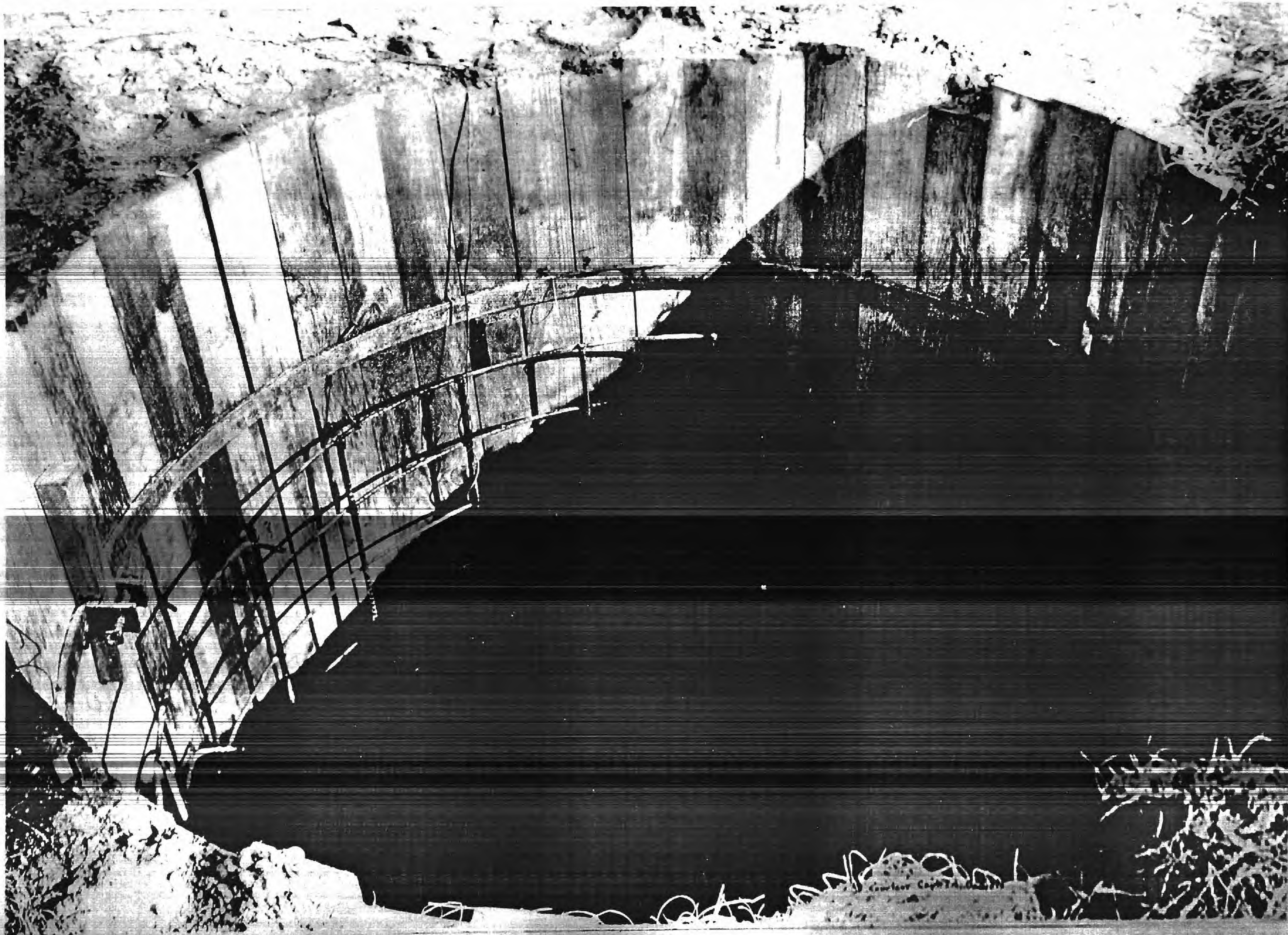
Open Pits.

When rock is found above the ground water level and the ground is both cohesive and dry, openspits may sometimes be dug for a distance of from ten to fifteen feet with out the use of sheeting or other protection of the sides.

Wood Sheet Piles.

It is very rare indeed to find ground conditions such as the above in actual practice. The ground is usually moist and non-cohesive; so that the sides of the excavation must be supported. In dry material, the planks are merely driven into the ground vertically and close together and if necessary braced. A very effective form of sheet piling is that called Wakefield sheet piling and is made by nailing or bolting three boards together, the middle plank being placed so as to form the tongue and groove. Tongue and grooved sheet piling must be used where the ground is not dry. In this type of excavation, the sheeting should be supported by wales which a usually 6 in. x 6 in. or larger and spaced vertically 6 to 7 feet apart near the top and closer together at the bottom. These wales are supported by horizontal struts running between opposite sides. Excavations made in this manner may be carried to a depth of 60 feet. However, when the depth is more than 20 feet, the width at the top is widened so as to allow another row of piling and wales to be placed inside the upper pit for every 20 feet of depth and have the width required at the bottom of the excavation.

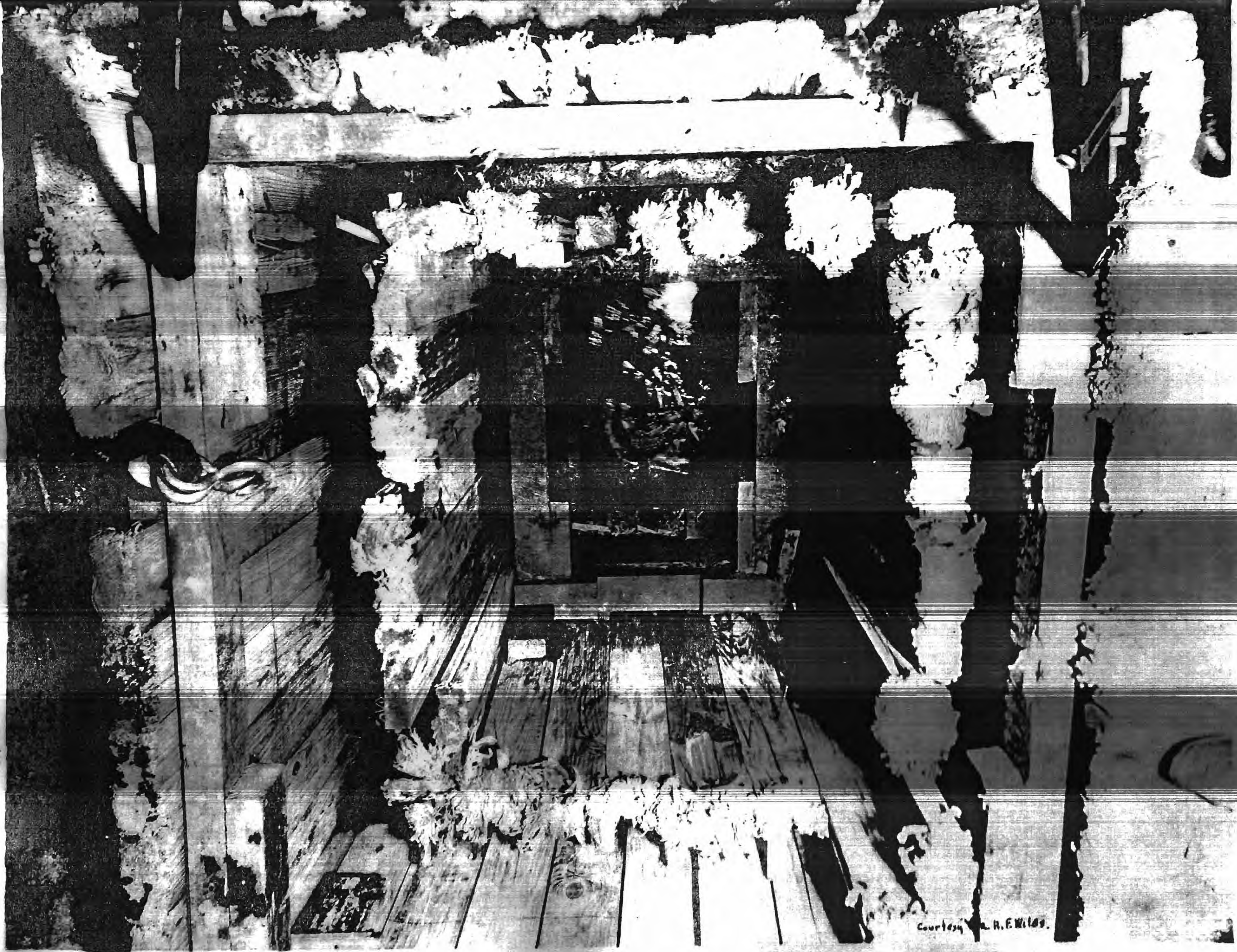
General Layout of Caissons
Atlanta Post Office



OPEN CAISSON 90 FT. TO ROCK

The Chicago Well Method.
When the material to be excavated is fairly stiff clay, the excavation may be carried down to depths of 100 feet or more by this method. This method being used on the Cleveland Terminal to take the foundations down 290 feet to rock, the deepest foundations yet constructed. In the Chicago Well method, the excavation is carried down by digging a circular well which ranges between 3 and 8 feet in diameter, according to the load carried; and in sections of about 5 feet in length. The sides of the excavation are protected by staves 1½ to 3 inches thick and about 6 inches wide and having a length of 5 feet. Each section of lagging is held in place by two steel rings 3 inches wide and about 3/4 of an inch thick, each ring consisting of two semi-circles flanged at the ends and connected by bolts. The iron rings are removed as the finished well is filled with concrete, but the lagging is allowed to remain and act as a form. These columns as they may be called are designed as such and reinforced in the same manner all according to the load carried. The bottom of these wells is usually belled out to from 1½ to 2 times their diameter. This method is especially adapted to and has been used extensively around Chicago and for that reason it has been so named.

The Gow Method.
In stiff clay or a plastic water bearing material, the Gow method is sometimes used for sinking foundations. It consists of carrying a single cylindrical concrete cylinder through unsuitable upper strata to a suitable



Open Well Excavation

bearing strata below. This is done by sinking a series of short steel cylinders, varying in diameter so as to telescope into one another, the largest size being used at the top and the others inserted successively through those already in place. When the desired strata is reached, the bottom of the strata is belled out to the desired diameter. The opening is then, entirely filled with concrete, the several steel cylinders being successively withdrawn as the concreteing progresses. The minimum area of these cylinders at the bottom is such as to satisfactorily transfer the load the the strata. The minimum diameter of any cylinder is 3 feet.

<u>Shafts</u>.
This method may be used where the ground is not too soft and not too water-bearing. It consists of constructing a hollow cylinder built of reinforced concrete, brick, wood, or a combination of the above, being sheathed on the sides with boards, and provided with a steel cutting edge at the bottom. The shaft is constructed to a conveneient height at the site and is sunk by excavating inside and under it. The weight of the shaft is used to overcome the friction on the sides and when it ceases to be sufficient, weights must be used.

<u>Steel Sheet Piling</u>.
Steel sheet piling may be used to take the place of tongue and grooved wood sheet piling where much ground water is encountered. The sections most used in land work are Carnegie Sections 104 and Bethlehem Sections SPB 12 and SPB 14.

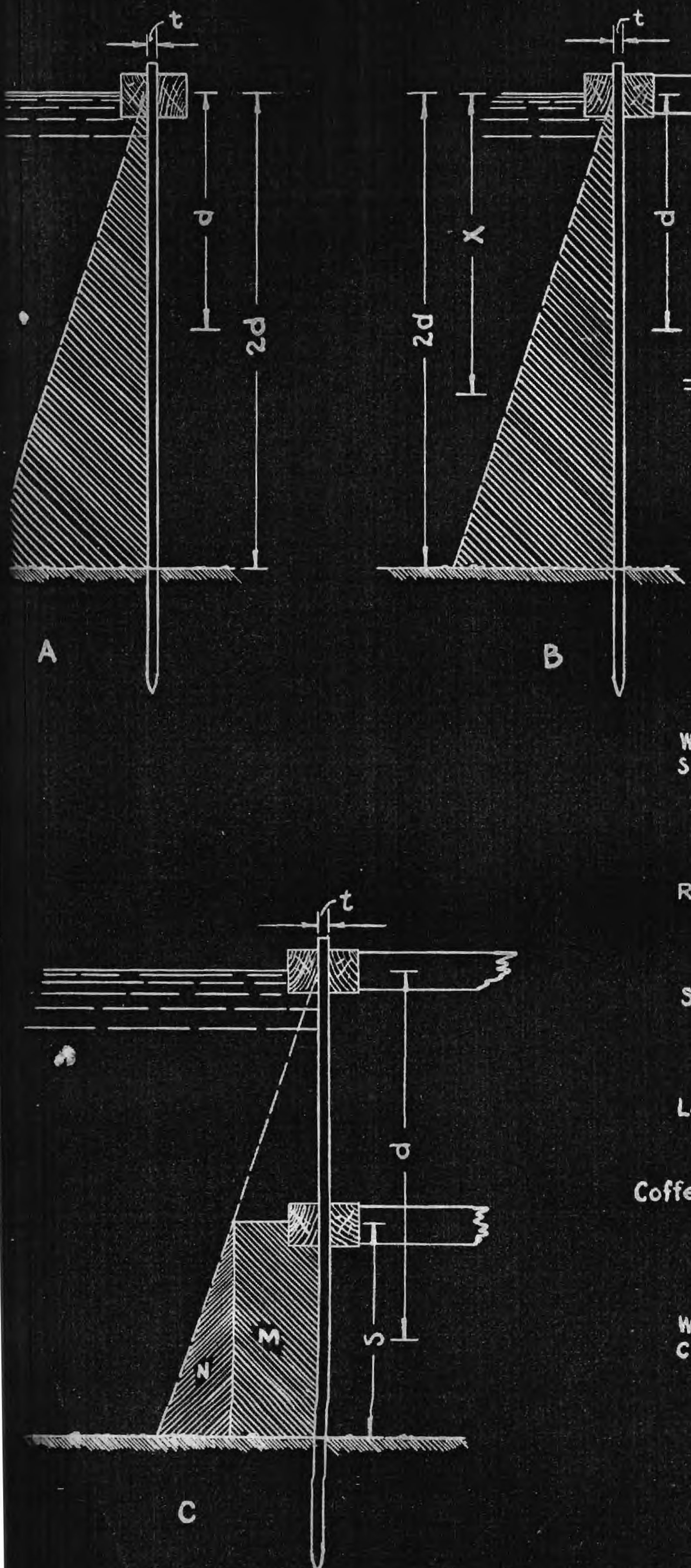


FIG. No.1
COFFERDAM CONSTRUCTION

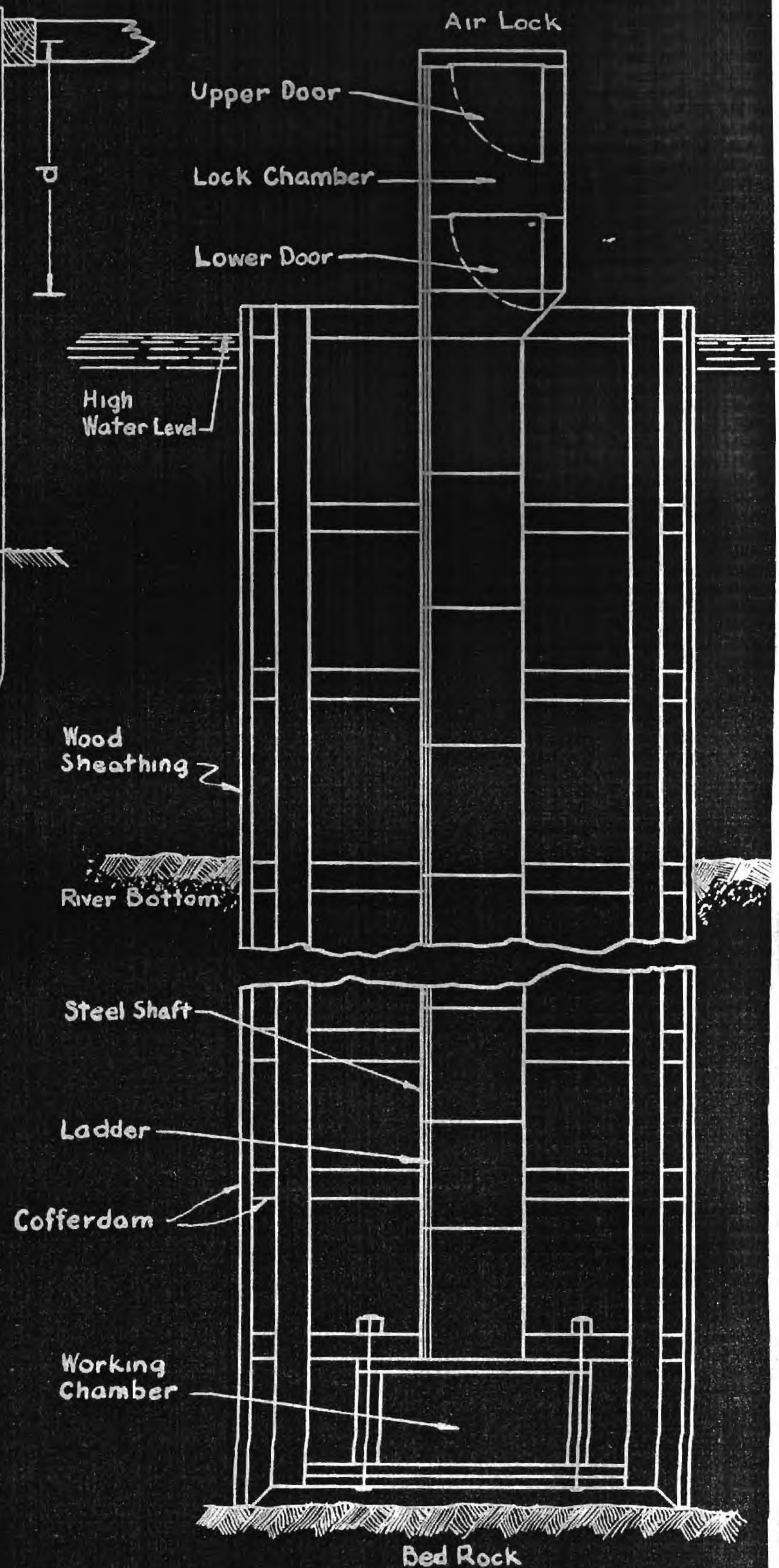


FIG. Nº 2
PNEUMATIC CAISSON

Design of Sheet Pile Construction in Water.

Engineers often fail to make proper provision for the pressure of the water against the sides of cofferdams in an accurate manner. Too often they depend upon past experience in determining the thickness of sheet piles and the distance between and the size of guide piles and wales.

The cofferdam may be built in such a manner as to act as a cantilever, a beam supported at both ends, or a beam over three or more supports depending upon the manner of construction and the number of wales used.

In the first, the sheet piles are actually in the condition of a beam fixed at one end and loaded with a gradually increasing weight due to the pressure of the water, whose weight is taken as 62.4 pounds per cubic foot.

The load on the width, w, of the wall whose heighth is 2d, is $124.8\ wd^2$ and the moment of the pressure is $83.2\ wd^3$. Taking the allowable unit stress on wet timber such as Southern Pine, Douglas Fir, Oak, and good grades of Cypress as 1,000 pounds per sq. in., the thickness, t, of the sheet piling = $(.496d^3)^{\frac{1}{2}}$ in which d is in feet and t in inches.

The addition of a strut at the top of the sheet piling as in Fig. 1(b) Plate No. 11 makes the sheet piling act as a beam supported at the upper end and fixed at the lower end, for simplicity, we calculate it as supported at both ends. The load will be the same as before, $124.8wd^2$. The maximum moment will occur at a distance from the top, $X = 1.16\ d$ and has a value of $32wd^3$.

The thickness of the sheet piling = $t = (.192d^3)^{\frac{1}{2}}$

If the section of sheet piling to be calculated is located as at S in fig. 1(c) Plate No. 11, the piling is in the condition of a beam supported at both ends and loaded with a uniform load M and a triangular load N. For practical purposes, we may consider all the load as uniform and due to the head acting at the middle of this span. This will give a load of 62.4wdS on the span S, for a width, w. The moment will be $7.8dS^2$, and with an allowable stress of 1000 pounds per sq. in., we have $t = (.047dS^2)^{\frac{1}{2}}$.

In the spacing of struts or braces, p = the allowable stress in pounds per sq. inch, l is the unsupported length in inches, and d is the least side of the timber in inches.

$p = 600 - 7(l/d)$

Methods Used in Water.

Cofferdams.

In this method theysite to be unwatered is enclosed with some material which is water-tight and will sustain the outside pressure of the water after the inside has been unwatered. In water which is very quiet, 4 Or 5 feet½deep and has a firm impervious bottom, this may be done by closing the site with sand and clay filled bags, or a dike consisting of two walls made of the bags and between tem filled with clay.

Puddle cofferdams are made by driving two lines of sheet piling to retain the puddle which is placed between them. Timber piles and wales are first driven to guided the sheet piles. The puddle should never be less than 3 feet thick. This type is best used where the bottom is stiff impervious clay. Most cofferdams now are built by driving a single row of steel sheet piling except where the bottom is bare rock. The sections most used for this work are Bethlehem Sections DP165 and SPE 14, the Larssen Section II, and the Jones & McLaughlin Section C-27. Steel sheet piling are usually driven between guided by a single or double acting steam or air hammer. It has been common t o build cofferdams to depth of 45 feet with steel sheet piles. Great care should be used to see that they are properly braced when carried over 10 feet in depth. The essential quality of any sheet pile structure in water, is that the strata into which the pile is driven, is impervious engough to resist the flow of water to a considerable depth below the proposed bottom. It is not possible to entirely eliminate leakage in cofferdams and pumps must be maintained to keep them from becoming flooded.

Floating Caissons.

This method is very old and under some conditions still very useful. It consists of sinking a floating box in which masonry is built asthe box sinks, the weight of the masonry overcoming the buoyancy of the box, thus the pier is built up until the bottom of the box finally rest upon the place previously prepared for it. The bottom of these boxes are usually made of reinforced concrete and the sides of timber construction.

Dredging Caissons.

When it is necessary to excavate to great depths under water, this method is used; as it can be carried to far greater depths than would be possible with a pneumatic caisson. A dredging caisson is simply a large box or shaft built of wood, steel, or reinforced concrete, and provided with one or more wells through which the material may be dredged. The caisson sinks as the material below is dredged out and as the compartments are filled with concrete whose weight overcomes the friction on the caisson walls. The caisson is built up as it sinks and is usually designed so its weight will overcome both its buoyancy and friction on the surrounding material without the use of additional weights as used in sinking drop shafts and pneumatic caissons. The friction to be overcome in this method is greater than that for pneumatic caissons. On the bridge across the Ganges at Sara, India the piers were taken down 185 feet below hggh water by this method.

Pneumatic Caissons.
This method is used where the intensity of the load is so great as to prohit the use of piles, where erratic boulders would prevent the penetration of piles to the desired stratum, where the depth of the bearing strata is so great as to prohibit the economic use of a coffer-dam, when it is necessary to carry the foundation throuh material which will flow and upon which rests nearby structures. This method is therefore widely lused in the construction of deep foundations for heavy buildings, bridges, and dams. Because of its high cost, it is used only where no other method can be applied. Foundations may be constructed by this method to a depth of from 100 to 120 feet below ground water or high water level. This method consists essentially of using compressed air to drive the water out of the space in which the men are working and alsoout of the voids in the material which is being excavated, so that it becomes comparatively dry and easy to handle, and will stand to a considerable height without lateral support. The excavation is done in a working chamber which is closed on the sides and top but open on the bottom and has a pressure within it greater than that of the water without. See Plate No. 11. Men work on the ground in this chamber and remove the material under the caisson sides causing it to sink. Access to the working chamber is provided by means of a cylindrical well or shaft leading from the working chamber

to a point above the water level. The shaft is provided with a double valve or air lock through which egress and ingress may be made without considerable loss of air, and also so that the air pressure on the workmen may be increased and decreased slowly so that they will not be so likely to have the "Bends". The supply of air at proper pressure is supplied by low pressure air compressors. Auxilliary compressors must be maintained. Caissons 5 to 7 feet in diameter may be sunk through from 5 to 7 feet of clay or silt in one 8 hour working day while larger caissons, 20 to 50 feet or more in diameter, may be sunk $1\frac{1}{2}$ to 3 feet in eight hours.

Table Number II. Areas and Summation of Perimeters also Weights of Bars

Summation of Areas = A. Summation of Perimeters = 0

Bar Size	Wt. Lbs.		Number of Bars 1	2	3	4	5	6	7
3/8"0	0.376	A	0.11	0.22	0.33	0.44	0.55	0.66	0.77
		0	1.18	2.36	3.53	4.71	5.89	7.07	8.25
1/2"0	0.668	A	0.20	0.39	0.59	0.79	0.98	1.18	1.38
		0	1.57	3.14	4.71	6.28	7.85	9.42	10.99
1/2"S	0.850	A	0.25	0.50	0.75	1.00	1.25	1.50	1.75
		0	2.00	4.00	6.00	8.00	10.00	12.00	14.00
5/8"0	1.043	A	0.31	0.61	0.92	1.23	1.53	1.84	2.15
		0	1.96	3.93	5.89	7.85	9.82	11.78	13.74
3/4"0	1.502	A	0.44	0.88	1.33	1.77	2.21	2.65	3.09
		0	2.36	4.71	7.07	9.42	11.78	14.14	16.49
7/8"0	2.044	A	0.60	1.20	1.80	2.41	3.01	3.61	4.21
		0	2.75	5.50	8.25	11.00	13.74	16.49	19.24
1" 0	2.670	A	0.79	1.57	2.36	3.14	3.93	4.71	5.50
		0	3.14	6.38	9.42	12.57	15.71	18.85	21.99
1" S	3.400	A	1.00	2.00	3.00	4.00	5.00	6.00	7.00
		0	4.00	8.00	12.00	16.00	20.00	24.00	28.00
1 1/8S	4.303	A	1.27	2.53	3.80	5.06	6.33	7.59	8.86
		0	4.50	9.00	13.50	18.00	22.50	27.00	31.50
1 1/4S	5.313	A	1.56	3.12	4.69	6.25	7.81	9.38	10.94
		0	5.00	10.00	15.00	20.00	25.00	30.00	35.00

Areas and Summations of Perimeters for Various Spacings.

Bar Size		Spacing of Bars in Inches 3"	4"	5"	6"	7"	8"	10"	12"
3/8"0	A	0.44	0.33	0.26	0.22	0.19	0.17	0.13	0.11
	0	4.71	3.53	2.83	2.36	2.04	1.78	1.41	1.18
1/2"0	A	0.78	0.59	0.47	0.39	0.34	0.29	0.26	0.20
	0	6.28	4.71	3.77	3.14	2.69	2.36	1.88	1.57
1/2"S	A	1.00	0.75	0.60	0.50	0.43	0.37	0.30	0.25
	0	8.00	6.00	4.80	4.00	3.43	3.00	2.40	2.00
5/8"0	A	1.23	0.92	0.74	0.61	0.53	0.46	0.37	0.31
	0	7.85	5.89	4.71	3.93	3.36	2.94	2.35	1.96
3/4"0	A	1.77	1.32	1.06	0.88	0.76	0.66	0.53	0.44
	0	9.42	7.06	5.65	4.71	4.04	3.53	2.83	2.36
7/8"0	A	2.40	1.80	1.44	1.20	1.03	0.90	0.72	0.60
	0	10.98	8.24	6.59	5.50	4.71	4.12	3.30	2.75
1" 0	A	3.14	2.36	1.88	1.57	1.35	1.18	0.94	0.78
	0	12.56	9.42	7.54	6.28	5.38	4.71	3.77	3.14
1" S	A	4.00	3.00	2.40	2.00	1.71	1.50	1.20	1.00
	0	16.00	12.00	9.60	8.00	6.86	6.00	4.80	4.00
1 1/8S	A	5.06	3.80	3.04	2.53	2.17	1.89	1.52	1.27
	0	18.00	13.50	10.80	9.00	7.72	6.85	5.40	4.50
1 1/4S	A	6.25	4.69	3.75	3.12	2.68	2.34	1.87	1.56
	0	20.00	15.00	12.00	10.00	8.57	7.50	6.00	5.00

1924
JOINT COMMITTEE SPECIFICATIONS APPLYING SPECIFICALLY TO FOOTINGS

Section 67. Metal reinforcement in wall footings and column footings shall have a minimum covering of 3 in. of concrete.

132. The shearing stress shall be taken as not less than that computed by the formula $v = V/bjd$. The stress on the critical section shall not exceed $0.02f'_c$ for footings with straight reinforcing bars, nor $0.03f'_c$ for footings in which the reinforcing bars are anchored at both ends by adequate hooks or otherwise as specified in section 140.

138. The permissible bond stress for footings and similar members in which reinforcing is placed in more than one direction shall not exceed 75% of $0.04f'_c$ for plain reinforcing bars and 75% of $0.05f'_c$ for deformed reinforcing bars.

140(b). Special anchorage shall be provided at the edges of footings, for all the bars for one third the working stress in tension shall be provided within a region where the tension in the concrete, computed as an unreinforced beam, does not exceed 40 lb. per sq. in.

172. The requirements for tension, compression, shear, and bond in sections 103 and 141, inclusive, shall govern the design of footings except as hereinafter provided.

173. The load per unit area on soil footings shall be computed by dividing the column load by the area of the base of the footing.

174. Footings on piles shall be treated in the same manner as footings on soil, except that the load shall be considered as concentrated at the pile centers.

175. Footings in which the thickness has been determined by the requirements for shear as specified in Sections 133 and 134 may be sloped or stepped between the critical section and the edge of the footing, provided that the shear at no section outside the critical section exceeds the value specified, and provided further that the thickness of the footing above the reinforcement at the edge shall not be less than 6 in. for footings on soil nor less than 12 in. for footings on piles. Sloped or stepped footings shall be cast as a unit.

176. The critical section for bending in a concrete footing which supports a concrete column or pedestal, shall be considered to be at the face of the column or pedestal. Where steel or cast iron column bases are used, the moment in the footing shall be computed at the middle and at the edge of the base; the load shall be considered as uniformly distributed over the column or pedestal base.

The bending moment at the critical section in a square footing supporting a concentric square column, shall be computed from the load on the trapezoid bounde d by one face of the column, the corresponding outside edge of the footing, and the portions of the two diagonals. The load on the two corner triangles of this trapezoid shall be considered as applied at a distance from the face equal to six-tenths of the the projection of the footing from the face of the column.

176 Cont. The load on the rectangular portion of the trapezoid shall be considered as applied at its center of gravity. The bending moment is expressed by the formula, $M = \frac{1}{2}w(a + 1.2c)c^2$, where M = bending moment at the critical section of the footing; a = width of face of column or pedestal; c = projection of footing from face of column; and w = upward reaction per unit of area of base of footing.

For a round or octagonal column, the distance a shall be taken as equal to the side of a square of an area equal to the area enclosed within the perimeter of the column.

177. The reinforcement in each direction in the footing shall be determined as for a reinforced concrete beam; the effective depth shall be the distance from the top of the footing to the plane of the reinforcement. The sectional area of the reinforcement shall be distributed uniformly across the footing unless the width is greater than the side of the column or pedestal plus twice the effective depth of the footing, in which case the width over which the reinforcing is spread may be increased to include one-half the remaining width of the footing. In order that no considerable area shall remain unreinforced additional reinforcement shall be placed outside the width specified, but such reinforcement shall not be considered as effective in resisting the calculated bending moment. For the extra reinforcement a spacing double that within the effective belt may be used.

178. The extreme fiber stress in compression in the concrete shall be kept within $0.04f'_c$. The extreme fiber stress in sloped or stepped footings shall be based on the exact shape of the section for a width not greaterthan that assumed effective for reinforcement.

179. A rectangular or irregularly shaped footing shall be computed by dividing it into rectangles and trapezoids tributary to the sides of the column, using the distance to the center of gravity of the area as the moment arm of the upward forces. Outstanding portions of combined footings shall be treated in the same manner. Other portions of combined footings shall be designed as beams or slabs.

182. The compressive stress in longintudinal reinforcement at the base of a column shall be transferred to the pedestal or footing by either dowels or distributing bases. When dowels are used, there shall be at least one for each column bar, andthe total sectional area of the dowels shall not be less than the sectional area of the longitudinal reinforcement in the column. The dowels shall extend into the pedestal or footing not less than 50 diameters of the dowel bars for plain bars, or 40 diameters for deformed bars.

When metal distributing bases are used, they shall have sufficient area and thickness to transmit safely the load from the longitudinal reinforcement in compression and bending. The permissible compressive unit stress on top of the pedestal or footing directly

182. Cont. under the column shall not be greater than that determined by the formula,

$$r_a = 0.25f'_c\sqrt[3]{\frac{A}{A'}}$$

where A = total area at the top of the pedestal or footing

A' = loaded area at the column base;

r_a = permissible working stress over the loaded area;

f'_c= ultimate compressive strength of concrete.

In sloped or stepped footings A may be taken as the area of the top horizontal surface of the footing or as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base the loaded area A', and having side slopes of 1 vertical to 2 horizontal.

183. The allowable compressive unit stress on the gross area of a concentrically loaded pedestal or on the minimum area of a pedestal footing shall not exceed $0.25f'_c$, unless reinforcement is provided and the member designed as a reinforced concrete column.

The depth of a pedestal or pedestal footing shall be not greater than three times its least width and the projection on any side from the face of the supported member shall be not greater than one-half the depth. The depth of a pedestal whose sides are sloped or stepped shall not exceed three times the least width or diameter of the section midway between the top and bottom. A pedestal footing supported directly on piles shall have a mat of reinforcing bars have a cross sectional area of not less than 0.20 sq.in. per foot in each direction, 3in. above the top of the piles.

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